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NATIONAL DAM SAFETY PROGRAM. GOSHEN DAM, INVENTORY NUMBER: VA-1--ETC(U)
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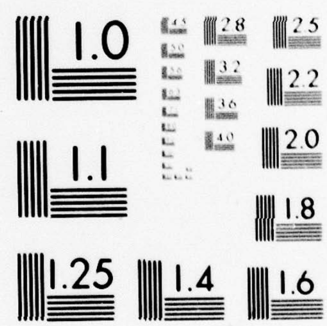

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Name Of Dam: GOSHEN DAM

Location: ROCKBRIDGE COUNTY, VIRGINIA

Inventory Number: VA. NO. 16301

LEVEL II

PHASE I INSPECTION REPORT

(6) NATIONAL DAM SAFETY PROGRAM.

Goshen Dam, Inventory Number: VA-16301. Rockbridge County, Virginia. Phase I Inspection Report.

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PREPARED FOR

NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY Richmond VA

SCHNABEL ENGINEERING ASSOCIATES, P.C.
J. K. TIMMONS AND ASSOCIATES, INC.

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

NAME OF DAM: GOSHEN DAM
LOCATION: ROCKBRIDGE COUNTY, VIRGINIA
INVENTORY NUMBER: VA. NO. 16301

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED FOR
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

SCHNABEL ENGINEERING ASSOCIATES, P.C./
J. K. TIMMONS AND ASSOCIATES, INC.

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of the Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the design flood should not be interpreted as necessarily posing a highly inadequate condition. The design flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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GOSHEN DAM
PHASE I - INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Goshen Dam, Va. No. 16301
State: Virginia
County: Rockbridge County
USGS Quad Sheet: Goshen
Coordinates: Lat. 37^{o} 57.57' Long. 79^{o} 27.5'
Stream: Little Calfpasture River
Date of Inspection: December 14, 1978

BRIEF ASSESSMENT OF DAM

Goshen Dam is a zoned earthfill structure 1300 ft long and 38 ft high (Elev. 1380). It includes a regulated reinforced concrete spillway near the west abutment. A 24' gravel roadway occupies the crest of the embankment and crosses the spillway via a steel bridge. The dam is located on the Little Calfpasture River, approximately 3 miles southeast of the community of Goshen, Virginia. The structure was designed by Herbert Associates of Harrisburg, Pennsylvania and constructed by English Construction company of Altavista, Virginia. The present owner is The National Capitol Area Council of the Boy Scouts of America. The upstream face of the berm is lined with rip rap and the downstream face was topsoiled and seeded. The spillway consists of a 158 ft wide by 18 ft high Ogee concrete spillway at elevation 1359.5 with regulated lift gates capable of increasing the lake water surface to Elev. 1969. The regulating gates increase the lake water surface elevation when they are in

the 'up' position and are lowered below the Ogee spillway crest when they are in the down position. The spillway has a 10" cast iron pipe system which allows a flow to be maintained in the downstream channel at all times. Also, there is a 3' x 4' sluice gate at the bottom of the outlet chamber to allow draining the lake. This impoundment is used for recreational purposes only, and during September through May the lake is kept at the level of the spillway crest (Elev. 1359.50 MSL). During the summer months, the large steel gates atop the spillway are in the 'up' position and the water level is raised to elevation 1369.0 MSL. The gates are motorized and operated automatically to maintain the lake at a constant elevation during periods of rainfall in the summer months. A control building located at the west end of the spillway contains the monitoring equipment.

The spillway is adequate to pass 48% of the PMF, which is approximately the intent of the original design. This dam is categorized as a "High" risk structure and is rated inadequate since the spillway will not pass the PMF and the dam would be overtopped by floodwaters from such a storm. The spillway is not seriously inadequate since it will pass 48% of the PMF.

The visual inspection revealed no apparent problems and there are no immediate needs for remedial measures. We do recommend that vegetation be conscientiously controlled. The downstream slope should be mowed several

times a year and existing small trees or saplings removed at least once a year. The actual embankment structure appears to be similar to the "design" drawings. The stability analysis of the embankment was reviewed and found to be somewhat inadequate, although factors of safety are acceptable. The slopes of the dam meet the requirements recommended by the U.S. Bureau of Reclamation for zoned earthfill dams and exhibit no indication of distress. The stability analysis of the spillway section was reviewed and both the method and the results appear reasonable.

Submitted:

Original signed by
JAMES A. WALSH

James A. Walsh, P.E.
Chief, Design Branch

Approved:

Original signed by

Douglas L. Haller

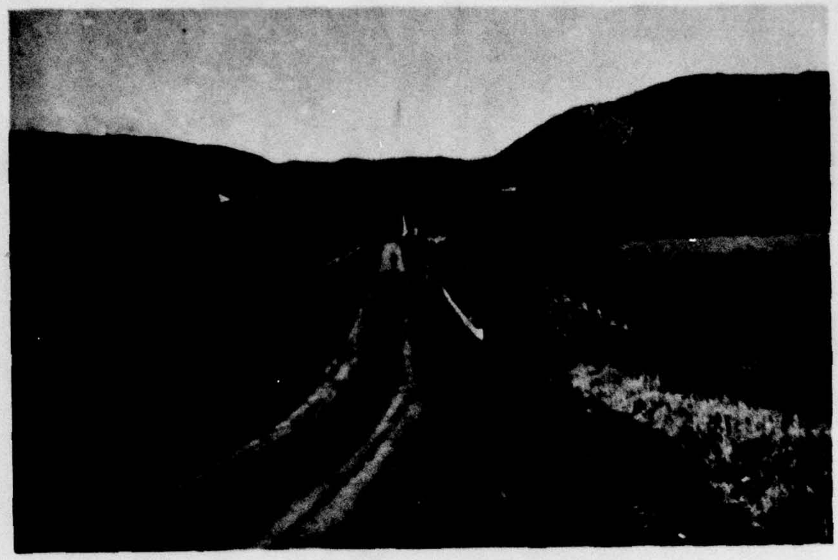
Douglas L. Haller
Colonel, Corps of Engineers
District Engineer

Recommended By:

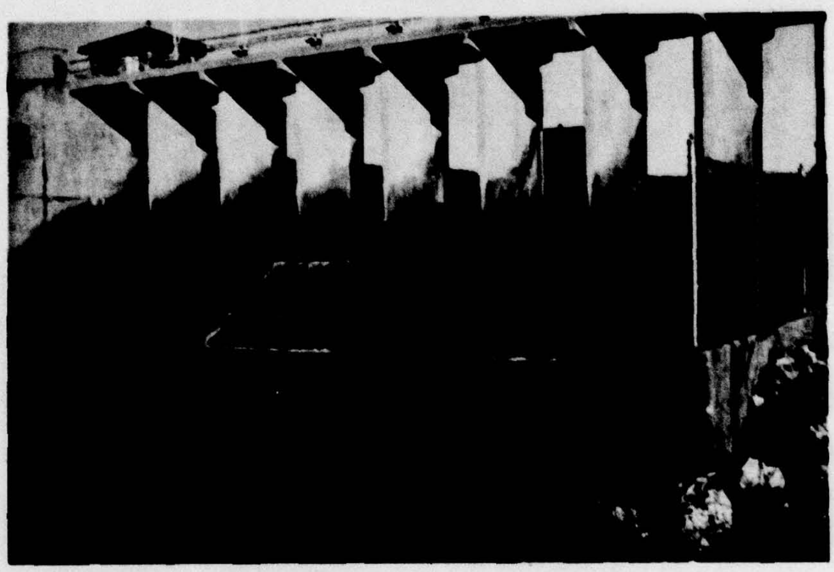
Original signed by
ZANE M. GOODWIN

Zane M. Goodwin, P.E.
Chief, Engineering Division

Date: MAR 15 1979



OVERALL VIEW - EARTHEN BERM
(VIEW FROM EASTERN ABUTMENT)



OVERALL VIEW - GATES AND SPILLWAY
(VIEW FROM DOWNSTREAM CHANNEL)

PHASE I - INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
GOSHEN DAM - VA. NO. 16301

SECTION 1 - PROJECT INFORMATION

1.1 General:

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (see Reference 1, Appendix V). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Goshen Dam is a zoned earth-fill structure (impervious core and semi-pervious shell) about 1300 ft long and 38 ft high. The top of the dam is 24 ft$\pm$ wide and is at El 1380.0 M.S.L. Side slopes are 2.5 horizontal to 1 vertical (2.5:1) on the downstream side and 3:1 on the upstream side. A 24' gravel road occupies the crest of the

embankment and crosses the spillway via a small steel bridge.

The reinforced concrete spillway consists of a modified gravity crest section 158 ft wide and sluice gate crest control and bucket-type energy dissipator. Concrete retaining walls extend beyond the downstream toe of the embankment on each side. The spillways empty into a riprap lined channel. The gate system is motorized and monitored via a control building located at the west end of the spillway. The building also has two 10" cast iron pipe inlets at its lower levels which are valved and used for low flow discharge. The lake can be drained through a 3' x 4' sluice gate located at the bottom of the control house. The sluice gate is connected to a 3' x 4' channel which discharges through the right abutment below the spillway. This gate is used only to drain the lake.

1.2.2 Location: Goshen Dam is located on the Little Calfpasture River about three miles southeast of the community of Goshen, Virginia. The reservoir formed by the dam is known locally as Merriweather Lake. and lies in a valley between Knot Mountain and Little North Mountain approximately 3000 feet north of State Rte 39. (see Sheets 1 and 2, Appendix I).

1.2.3 Size Classification: The dam is classified as an "Intermediate" size structure because of its maximum storage potential of 11,200 acre-feet at Elev. 1380 MSL.

1.2.4 Hazard Classification: Based upon the close proximity of the confluence with the Calfpasture River (1 mi.± downstream) and close downstream location (4 mi.±)

of Wilson Springs and Rockbridge Baths (population $100^{\pm}$), the dam is assigned a "High" hazard classification in accordance with guidelines contained in Section 2.1.2 of Reference 1, Appendix VI. The hazard classification used to categorize dams is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: National Capitol Area Council of the Boy Scouts of America.

1.2.6 Purpose: Recreation

1.2.7 Design and Construction History: The dam was designed by Herbert Associates of Harrisburg, Pennsylvania and constructed by English Construction Company of Altavista, Virginia. Construction was completed in 1966.

1.2.8 Normal Operational Procedures: Operation of the project is both automatic and manual. The spillway is gated, therefore water may be controlled and discharged downstream in desired quantities. The gates are raised during the periods in which the lake is to be used and lowered during the times that the lake is not in use and during flood conditions. At low flows, the 10" C.I. bypass is used to pass the required minimum flows.

1.3 Pertinent Data:

1.3.1 Drainage Areas: The dam controls a drainage area of approximately 85 square miles (Study by Mathew W. Delfert, Reference 7, Appendix V), which we have verified.

1.3.2 Discharge at Dam Site:

Maximum known flood - 7200 CFS (Estimate) in August 1969
Water Surface Elevation 1373

Principal Spillway - Pool level at top of Dam (EL 1380) with Gates below Spillway Crest - - - 42,000 CFS

Regulated 10" Low Flow Outlet - Normal Pool Level (Elev. 1369) - - - - - - 4 CFS(min)

3'x4' Drain Normal Pool Level (Elev. 1369) --380 CFS

1.3.3 Dam and Reservoir Data:

See Table 1.1

LAKE MERRIWEATHER
Table 1.1 DAM AND RESERVOIR DATA

Item	Elevation feet M.S.L.	Area Acres	Reservoir Capacity Acre Feet	Reservoir Capacity Watershed Inches (a)	Length Miles
Top of Dam	1380	700	11,200	2.47	3.3
Maximum pool, design surcharge	1377	625	9,000	1.99	3.1
Principal Spillway crest (b)	1369	438	4,672	1.03	3.0
Streambed at centerline of dam	1342				

(a) Based on 85 sq. mi.
(b) Cates in "Up" Position

SECTION 2 - ENGINEERING DATA

2.1 <u>Design</u>: The dam was designed by Herbert Associates of Harrisburg, Pennsylvania and constructed by English Construction Company of Altavista, Virginia. Contract drawings and reports are available from Mr. Elgin Deering, National Capital Area Council, Boy Scouts of America, Wisconsin Avenue and Cedar Lane, Bethesda, Maryland, 20014.

A subsurface investigation was conducted at the site by F. T. Kitlinski and Associates during the initial design stages. The investigation consisted of drilling 39 test borings as shown on Sheet 3, Appendix I. Test boring logs and associated subsurface profiles for borings made along the dam and spillway axis are shown on Sheets 4 and 5 of Appendix I, respectively. Laboratory test data is summarized on Sheet 7, Appendix I. The geotechnical report with recommendations was prepared based upon the test boring, permeability and laboratory test data.

The dam is a zoned, compacted-fill earth embankment consisting of an impervious clay core and semi-pervious or pervious shell. The upstream slope includes a riprap layer and a drainage blanket and toe drain extending along the downstream toe. Details are shown on Sheet 6, Appendix I. The following descriptions of embankment

fill materials were included in the construction specifications:

A. "The impervious compacted fill shall consist of native material removed from the impervious Borrow Area shown on the General Plan and used in the core of the dam. Results and extent of the investigation of this material are included in the "Report on Investigation of Foundation Conditions", available for inspection at the office of the Engineer or the Ranger's House. If the Contractor elects to use other impervious Borrow Areas within the lake, he shall pay the cost of all necessary tests of the new material to determine acceptability by the Engineer.

B. "The semi-pervious and pervious materials used in the shell of the dam shall be obtained from required excavations and Borrow Areas selected by the Contractor from within the lake area (below elevation 1369.0) and approved by the Engineer."

The construction specifications required that the fill be placed in 6-inch layers and compacted with a roller to 95 percent of maximum dry density in accordance with ASTM D-1557, Modified Proctor.

Design drawings show the dam being founded on overburden and include a 15 ft wide cutoff trench, which extends the length of the dam axis. This trench has 1:1 side slopes and is filled with impervious clay material. The spillway is indicated to rest on sound

shale bedrock at EL 1340$\pm$, which is the general elevation designated as the top of sound bedrock along the dam axis. Both abutments tie into sound shale.

Based upon the pressure test data, it was recommended that a grout curtain be constructed beneath the cutoff trench in order to control water seepage through the fractured rock. Grouting requirements are included in the construction specifications. To control the phreatic water surface and to collect seepages, a drainage blanket and a rock toe drain were constructed along the downstream portion of the dam. Details are shown on Sheet 6, Appendix I.

The dam includes a gravity-type, reinforced concrete spillway which is located near the right abutment. Design drawings show the spillway being founded on sound shale bedrock at EL 1340$\pm$. The crestline of the 158 ft wide spillway is at EL 1359.50 M.S.L. A sluice gate crest control system is used to control the lake level and the overflow enters a bucket-type energy dissipator.

A cursory design stability analysis was performed by Herbert and Associates using a stability number approach. The shear strength of the core material was determined by direct shear test. An angle of interior friction of 23^{o} and cohesion of 900 psf were used in the analysis.

Concrete retaining walls extend beyond the downstream toe of the embankment on each side.

2.2 Construction: The construction records were not included with the information provided by the Owner.

2.3 Operation: An Operations and Maintenance manual was prepared by Herbert and Associates which is used by the dam maintenance and operations personnel. The control building has full instrumentation for flow and control data.

2.4 Evaluation: The design calculations are adequate except for the stability analysis, and the design drawings are representative of the dam.

SECTION 3 - VISUAL INSPECTION

3.1 General: An inspection was made 14 December 1978 at which time the pool elevation was 1359.7 MSL (0.2 ft above the spillway crest) and all gates except one were in the down position (below spillway crest). The temperature was 35°, the sky was partly cloudy, and the wind was from the north at 5-10 mph.

3.2 Findings:

3.2.1 Dam and Spillway: There is small tree and tall weed growth on the downstream slope of the embankment. The cold joints in the second level of outlet chamber show some bleeding. The gates and motors appear to be kept in good operating condition except that one gate is non-operational. All concrete surfaces of the spillway appear to be in good condition. The flap valve on the left spillway wing wall which regulates flow into the downstream toe drain during periods of high flow is missing.

3.2.2 Reservoir Area: Shoreline is in good condition, however, debris has collected extensively along the upstream face of the dam.

3.2.3 Appurtenant Structures: Control house and operating valves were reportedly in operating condition.

3.2.4 Downstream Area: The Little Calfpasture River joins the Maury River immediately downstream of the dam. There are homes and cabins on the Maury River Floodplain within 4 miles at Wilson Springs. Goshen Pass immediately downstream of the confluence of the Calfpasture River and Little Calfpasture River is very restrictive to river flow.

3.3 Evaluation:

3.3.1 Dam and Spillway: Overall the dam was in good condition at the time of inspection. Uncontrolled growth encourages the development of deep rooted vegetation. This type of growth can encourage piping within the embankment. Also, excessive growth inhibits effective visual inspections of the dam. The downstream slope should be mowed at least once a year, but more preferably twice a year (before and after the summer months). If tree growth occurs on the slopes, these trees should be removed at the time of mowing. The concrete spillway and abutment walls show no deterioration. The flap valve on the left spillway abutment should be replaced.

3.3.2 Downstream Area: If a dam failure were to occur, Route No. 39 through Goshen Pass would probably be inundated due to the restrictive gorge, and the houses and cabin at Wilson Springs could be flooded.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: Lake Merriweather provides recreational needs for the surrounding property owned by The Boy Scouts of America. The lake is raised to elevation 1369 MSL (top of gates) during the months of June through August to provide maximum lake use. Normal flow is passed through the dam by a regulated 10 inch intake which allows discharge equal to inflow, and stabilizes the lake elevation. During periods of high inflow, the crest gates automatically keep the lake at a constant elevation by allowing flow over the principal spillway. During the months of September through May, the lake is maintained at elevation 1359.5 MSL with the crest gates lowered below the spillway crest. Flow is through the 10 inch intake and through the principal spillway as the inflow rate dictates. These procedures are outlined in the Operations and Maintenance Manual prepared by Herbert Associates, Inc.

4.2 Maintenance of Dam: Debris is periodically removed from the floating log fence as required. Removal of debris from the upstream dam face and mowing of the downstream dam are routine maintenance requirements.

4.3 Maintenance of Operating Facilities: A maintenance checklist and procedures are indicated in the Operations and Maintenance Manual provided by Herbert Associates, Inc.

The following items are to be checked according to the times indicated.

Item	Frequency of Inspection
Concrete Surfaces . . .	Semi-annually
Main Drain Gate	Only when lake is drained
Crest Gates	Semi-annually

In 1976 the spillway crest gates were repainted.

4.4 Warning System: The crest control system consists of 10 individual steel gates, each with rubber seals at the bottom and sides to prevent leakage. Automatic and manual controls are provided to adjust the crest level at any desired height and/or to open fully to pass high flood discharges. In its fully opened position, the system conforms to the shape of the overflow section, presenting an opening with a high flow coefficient. The crest gates have an automatic control system that, if the water level varies more than 15" either above or below normal pool, sets off a horn sounding an alarm and automatically at set times raises or lowers the gates to control the water level.

4.5 Evaluation: The operation and maintenance appears to be good and the equipment is kept in good operating condition. The procedure outlined in the manual prepared by Herbert Associates has been reviewed and found satisfactory for this type of impoundment. This manual should be followed for the operation and maintenance of this facility. Weekly and monthly operating reports were prepared for use by Operator and should be kept current at all times. These offer a

good evaluation of facilities.

SECTION 5 - HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: The Merriweather Lake Dam was designed under the authority of The Boy Scouts of America for recreational and aesthetic purposes. The normal pool elevation (1369) was established at an elevation which would maximize recreational use of the lake. The principal spillway has a gated crest with an elevation of 1359.5 MSL. The elevation of the spillway crest was established by the designers on the basis of the crest gates down and the spillway passing the ½-PMF, which they determined was 45,000 CFS (inflow). The top of dam elevation (1380) was established to allow a roadway to be built across the top of the spillway without interfering with the design discharge at a pool elevation of 1377.

5.2 Hydrologic Records: None

5.3 Flood Experience: The maximum pool elevation recorded was elevation 1373 in August 1969 as a result of rainfall from Hurricane Camille. This pool elevation was 4 ft above the top of the crest gates.

5.4 Flood Potential: The Probable Maximum Flood (PMF), ½-PMF, and 100-year Flood hydrographs were developed by the SCS method (Reference 4, Appendix V). Precipation amounts for the flood hydrographs of the PMF, ½-PMF, and 100-year Flood were taken from the U.S. Weather Bureau information

(References 5 & 6, Appendix V). Appropriate adjustments for basin size and shape were accounted for and hydrograph determination procedures as outlined in Reference 4, Appendix V, were used for the flood hydrographs. These hydrographs were routed through the spillway to determine maximum pool elevations.

5.5 <u>Reservoir Regulations</u>: Regulation of the reservoir level is automatic when the crest gates are up. Normal water flow is passed through two 10-inch screened intakes at Elevations 1361 and 1350.5, respectively, and the crest gates are in the raised position. During periods of rainfall the crest gates open automatically to allow water to pass over the spillway and maintain the lake at normal pool elevation (1369). The normal pool elevation is maintained during the summer months (June through August) and during the balance of the year the lake is maintained at the spillway crest elevation (1359.5). The crest gates are manually operated during the period when the gates are down (Pool Elevation 1359.5).

Routing of the PMF and ½-PMF began with the pool elevation at 1369 since this represents a condition when the spillway crest gates are up and available storage is at a minimum.

Stage-Discharge and Stage-Storage data used in flood routing were obtained from reports and computations provided by the owners' consultant. For flow rates higher than those provided, values were obtained by extrapolation of the data.

5.6 Overtopping Potential: The probable rise in the reservoir for the PMF, ½-PMF, and 100-year Flood, and other pertinent data on reservoir performance in various hydrographs, is shown in Table 5.1.

Table 5.1 RESERVOIR PERFORMANCE

	Normal Flow	HYDROGRAPH		
		½ PMF	PMF	100-Yr.
Peak flow, CFS				
Inflow		50,352	104,900	19,898
Outflow		42,743	105,027	25,137
Maximum Elevation, M.S.L.	1369	1380.07	1385.63	1374.11
Spillway (Elev. 1359.5) (Gates Below Crest)				
Depth of flow, ft		20.57	26.1	14.61
Duration, hours		1.5	2.0	2.0
Velocity, fps		16.0	18.4	13.2
Non-overflow Section				
Depth of Flow, ft		N/A	5.63	N/A
Duration, Hours			2.0	
Velocity, fps			6.3	
Tailwater elevation, M.S.L.	1342.5(a)	1351	1361(b)	1349

(a) Elevation when observed. This corresponds to normal flow.
(b) Controled by Goshen Pass on the Maury River.

5.7 Reservoir Emptying Potential: The 3' x 4' sluice gate at the upstream, bottom side of the outlet tower will pass an average of 158 CFS with the pool elevation at 1359.5. Assuming an average inflow of 5.8 CFS (based on average low flow stream data), this will dewater the lake in approximately 6 days.

5.8 Evaluation: Hydrologic and hydraulic determinations of the dam as designed appear reasonable. The dam and spillway were designed to accommodate the ½-PMF and under Corps of Engineers' standards, the spillway will pass 48% of the PMF. The dam is overtopped during the PMF. Present day conditions of the drainage basin were used in determining the hydrology used in evaluation of the dam.

SECTION 6 - DAM STABILITY

6.1 Foundation and Abutments: The dam site is located within the Valley and Ridge Physiographic Province of Virginia, which is underlain by sedimentary rocks from Middle Cambrian through Early Mississippian Age. In the Lexington area, the province consists of the Shenandoah Valley to the east and a series of much narrower valleys and intervening ridges to the west. The eastern portion of the province includes southeastward dipping thrust faults and asymetric folds, which are overturned to the northwest. More open folds are common in the central and western areas. Most ridges are "held up" by sandstones and conglomerates, whereas valleys are underlain by less resistant shales and limestones.

The Goshen Dam - Lake Merriweather complex is underlain by the Millboro Formation of Middle Devonian geologic age (see Reference 3, Appendix V). The Millboro is approximately 1000 ft thick and consists of black fissile shale, which usually weathers to a light gray and gray-brown color. On a typical outcrop, the shale is broken up into thin slivers and flakes. The dam and its impoundment rest along the axis of the Little Calfpasture syncline, which strikes approximately 45^{o} northeast. Bedding dips reportedly range from 15 to 40 degrees to the southeast and northwest, however, folding

within the syncline results in the local steepening of beds to the vertical position. Attitudes measured on shale outcrops in the right abutment indicate bedrock strikes from 58 to 76^{o} northeast and dips 34 to 48^{o} southeast. Bedding dips steepen to the vertical position locally. Jointing was also observed and includes essentially rectangular joint sets. No faults were encountered in the field during this investigation and geologic maps of the area do not show the presence of faults in the immediate vicinity.

Excluding the left abutment, natural soils encountered in the subsurface investigation along the dam axis and spillway are composed of two general types of material; an upper stratum of silty clay (CL) and clayey silt (ML) possessing low rates of natural permeability and an underlying stratum of silty sand (SM) and clayey sand (SC) having a high permeability rate. A special condition exists along the left abutment where the upper stratum of CL-ML soils is underlain by approximately 10 to 25 ft of large cobbles and boulders in a silty to clayey sand matrix. Only a thin wedge of the upper stratum soils (CL-ML) were encountered in the right abutment area, which consists of a moderately steep to steep weathered shale slope. Overburden thicknesses encountered along the

dam axis and spillway range from 3 to 25 ft and average approximately 13 ft. The underlying bedrock is described as being "sound" except for the top 2 to 3 ft which is usually weathered and moderately to highly fractured.

The geotechnical consultant reported that the abutments on both the east side and the right side of the dam appeared to have good shale to tie into. Prior to construction, the existing ground surface in the right abutment rose sharply from El 1350$\pm$ to 1416$\pm$ in 100 ft and it was expected that the abutment would have to be carried nearly 50 ft into the hillside before hitting sound rock. The greatest depth of overburden exists on the left side of the dam, where the topography of the area rose more gently. It appeared that the abutment would have to be carried to Station 19+00, or perhaps to Station 21+00, in order to tie into sound rock. The cut-off trench excavation on the east side was expected to be deeper than that required along the main portion of the dam due to the deep deposits of cobbles and boulders. It was believed that the excavation might go as deep as 25 ft before reaching sound rock. Visual inspection of both abutments confirmed the above physical descriptions. The left abutment consists of a steep hillside which includes numerous weathered shale outcrops. The left abutment is comprised of gentle slopes and only a few small, highly weathered shale outcrops were located.

The following recommendations were made by the geotechnical engineer for the design and construction of the dam such that settlement would be minimal:

1. "That all of the overburden soils, together with the weathered portion of the shale bedrock, be excavated to firm bedrock along the dam axis, including the abutments, and a cut-off trench of impermeable soils be constructed.

2. That the spillway section be located on sound bedrock.

3. That the existing virgin soils of the entire dam area, outside of the cut-off trench sector, upon which the embankment will rest, be proofrolled with a 50-ton pneumatic tired roller before receiving any fill materials, so as to increase the soil-supporting and impermeability characteristics of the soil strata. Any soft areas disclosed by this rolling should be removed and replaced with suitable material."

The geotechnical consultant recognized the potential for water seepage through two mediums. The silty sand strata, which possessed a high rate of permeability, was described as an undesirable part of the dam foundation because of its vulnerability to underseepage, excessive water loss and piping under the main portion of the dam. In view of this condition, it was recommended that the existing overburden soils be excavated to sound bedrock

along the dam axis, and that a cut-off trench (using a base of about 14 ft and side slopes of 1:1) of impermeable soils be constructed to restrict seepage.

Pressure tests performed in the underlying shale bedrock for design indicated a permeability value of 10^{-3} cm/sec or pervious condition according to the Bureau of Reclamation guidelines. Water loss was generally attributed to fractures and the dissolution of calcite layers within the bedrock. The geotechnical report recommended: "that a systematic program of grouting the rock foundation be developed and carried out at this site to improve the water holding capacity of the foundation." Grouting under pressure to seal seams, joints, and other openings is usually carried to a depth below the rock surface equal to the reservoir head above the surface of the bedrock. Design considerations indicate that a maximum elevation for the lake level would be near El 1370. On this basis, with the average rock surface at El $1340^{\pm}$, the grouting operations were recommended to a depth of about 30 ft below the rock surface or to El $1310^{\pm}$, following a split spacing, stage grouting zone method (15 ft per zone). Depending on the nature of the rock surface after excavation to the foundation base, a grouting cap approximately 3 ft x 3 ft in dimension was recommended to facilitate the subsurface grouting.

6.2 Embankment: The upstream slope is 3 horizontal to 1 vertical with crest at El 1380. A typical section of the dam is included on Sheet 6, Appendix I. At El 1358.5 the slope flattens to a level surface, forming an 8 ft wide berm. The slope then continues at 3 horizontal to 1 vertical to natural ground at El 1350±. The slope is blanketed with 24 inches of riprap extending from the top of the slope to its junction with the berm. The downstream slope is 2.5 horizontal to 1 vertical and terminates with a crushed stone toe drain containing a 6" diameter perforated CMP at the base. A 12 inch crushed stone drainage blanket encased in a 12" sand filter extends from a line 35 ft from the centerline to the toe drain. The toe drain terminated at El 1357 in the left abutment and at El 1357 in the left abutment. The sloping core consists of impervious clay (essentially compacted CL material according to ASTM D-2487) with 2/3 horizontal to 1 vertical slopes on the upstream side and 1/2 horizontal to 1 vertical slopes on the downstream side. The core ties into the underlying core trench with 1 horizontal to 1 vertical slopes. The upstream shell is constructed of semi-pervious fill, while the downstream shell consists of semi-pervious or pervious fill. This outer shell fill consists of material classified SM, SC, and SP. Physical properties of the core materials are summarized on Sheet 7, Appendix I.

6.3 Evaluation:

6.3.1 Foundation and Abutments: Dams must be evaluated on the basis of potential settlement, sliding, and seepage. Excessive settlement of the dam is not believed to be a problem based upon the information presented in the geotechnical report. From a strength or bearing capacity standpoint, the natural soils are satisfactory. Gradual consolidation of these soils had probably fully occurred under the applied load at the end of the construction period. The underlying shale bedrock is fairly competent and was assigned an allowable bearing value of 20 TSF, exclusive of the upper weathered and fractured zone.

Sliding within the dam foundation does not appear likely. A review of the geologic data and field examination indicates that there are probably no adversely oriented weak planes within the foundation rock that would act as a potential sliding plane.

Review of construction specifications and design drawings indicates that a core trench was planned and an extensive grouting program initiated as recommended by the geotechnical engineering report. This solution is considered adequate for limited underseepage through the dam foundation.

6.3.2 Embankment: The embankment slopes do meet the requirements recommended by the U. S. Bureau of Reclamation for small zoned earthfill dams on stable foundation. The Stability Analysis is included in Appendix IV. Since no undue settlement, cracking, or seepage was noted at the time of inspection, it appears that the embankment is adequate for normal pool operation with water level at El 1369.

The strength parameters described in Section 2 were used in the Stability Analysis. The analysis was performed using consolidated undrained direct shear or R-tests as defined in Reference 1, Appendix VI. These data represent only the impervious core soils. No testing of the semi-pervious to pervious shell material was performed. The factor of safety for the upstream slope under sudden draw-down is 2.59 as given in Appendix IV. A factor of safety of 6.49 was determined for a saturated condition which does not conform to a steady seepage analysis and utilized a 3:1 downstream slope instead of the 2½:1 slope which was constructed. The analysis is of questionable value, however, Herbert and Associates indicated that the dam design was reviewed by the U. S. Army Corps of Engineers, Washington, D.C., and was approved for construction informally as a courtesy to the Boy Scouts of America. The factors of safety do meet the requirements of Reference 1, Appendix VI.

The Stability Analysis of the spillway section was reviewed and found to be acceptable both as to method and factors of safety obtained.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment: The Goshen Dam as inspected 14 December 1978, appears sound and in a safe operating condition. The spillway will pass 48% of the PMF without overtopping the roadway along the top of the dam. Therefore, the spillway is considered inadequate because it is a "High" risk category dam. The spillway is not considered seriously inadequate since it will pass 48% of the PMF.

Based on the visual inspection and review of existing records, there is no apparent problem that would require immediate action for the normal pool conditions. The actual embankment structure appears to be similar to the "design" drawings. Without the construction records, the conformance of the embankment material properties to design requirements cannot be assessed. The embankment slopes meet the requirement recommended by the U. S. Bureau of Reclamation, Reference 2, Appendix V, for small zoned earthfill dams on stable foundations, and additional stability analysis is not considered necessary.

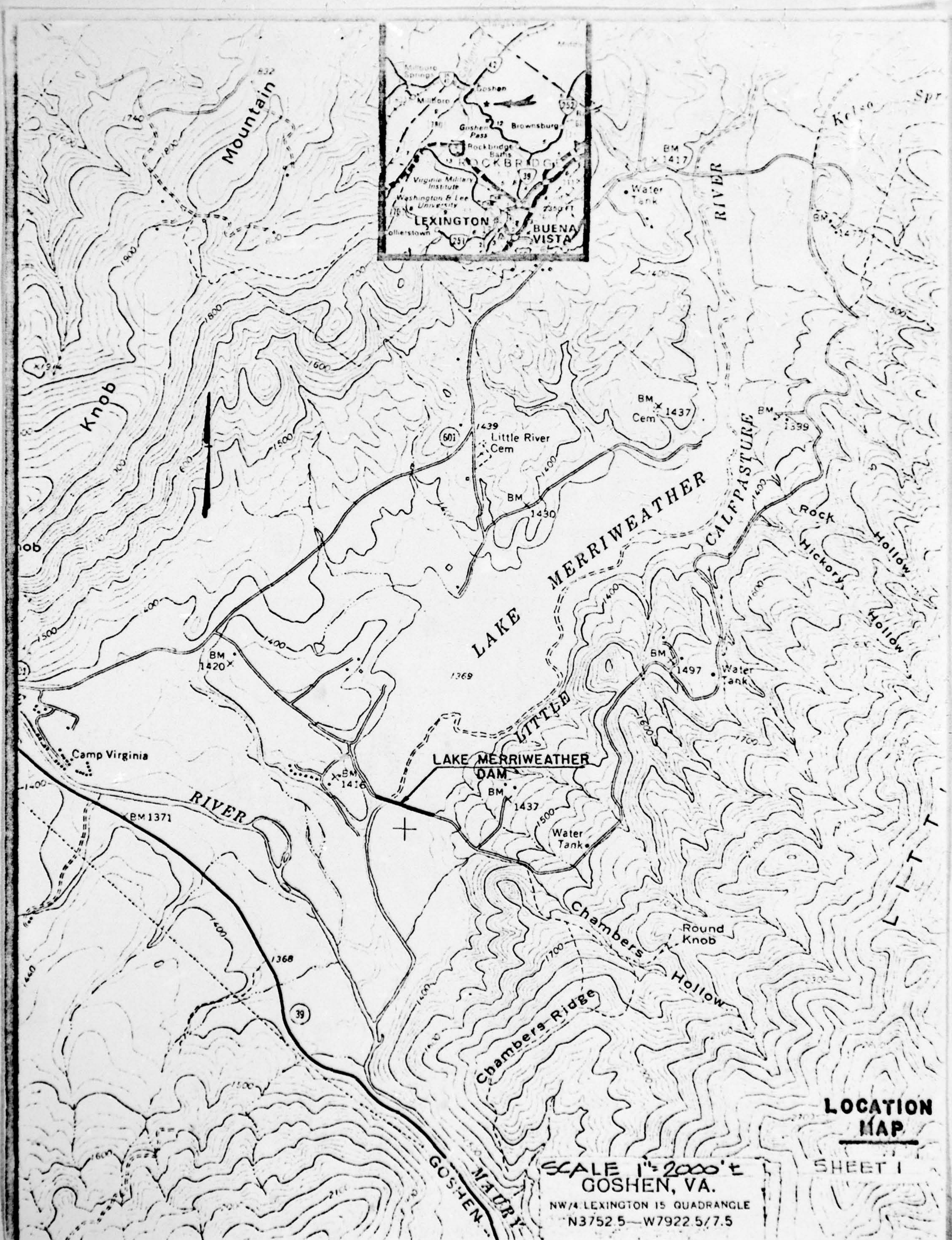
7.2 Remedial Measures: There is no immediate need for remedial measures; however, the following maintenance is suggested and should be initiated within 3 months. These measures are suggested for monitoring and maintenance purposes only.

7.2.1 <u>The grass and weeds along the downstream slope</u> should be cut at least once and preferably twice a year. Maintenance is recommended in the early summer and fall.

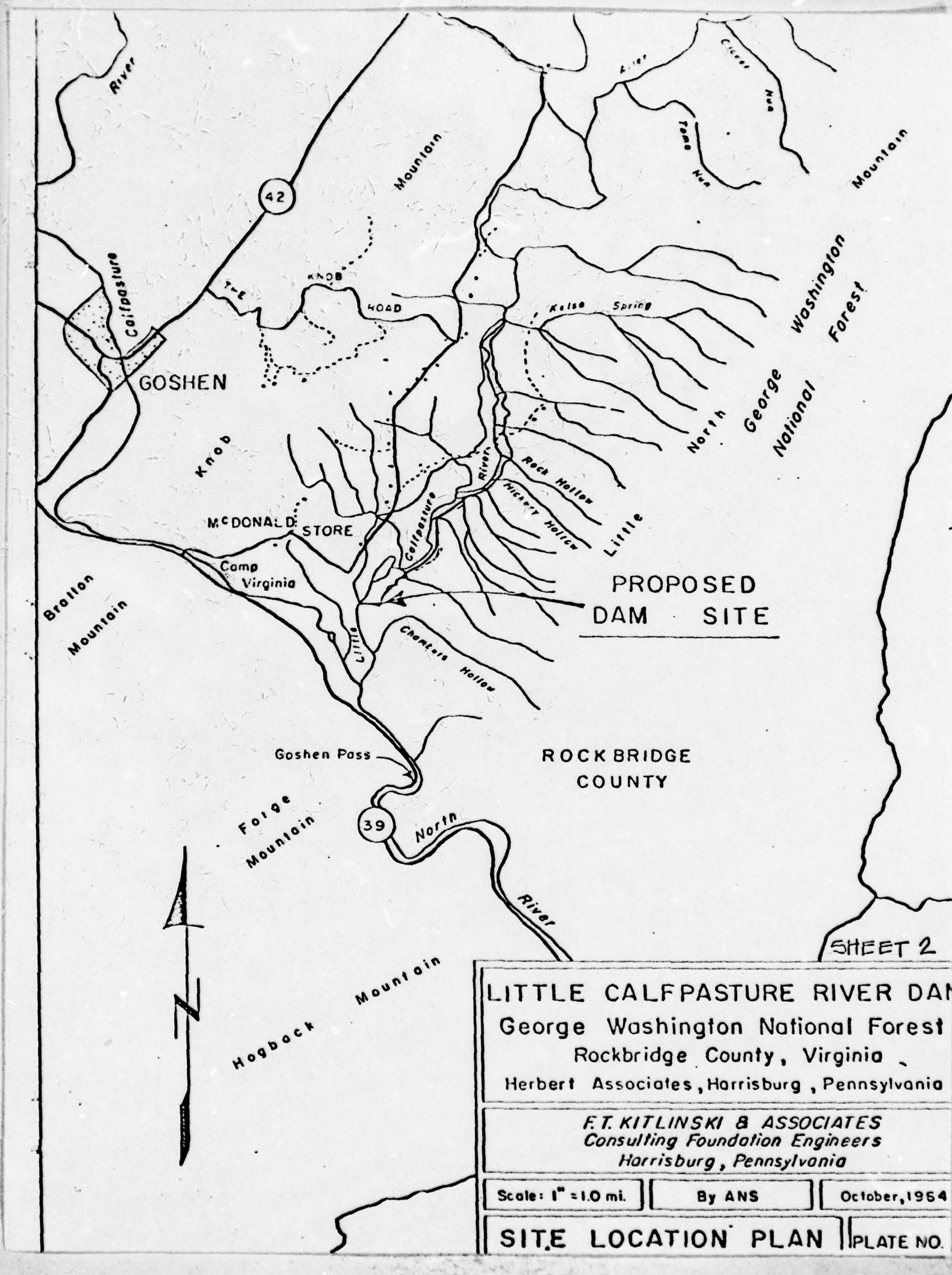
7.2.2 <u>The flap valve</u> on the east abutment toe drain should be replaced.

APPENDIX I

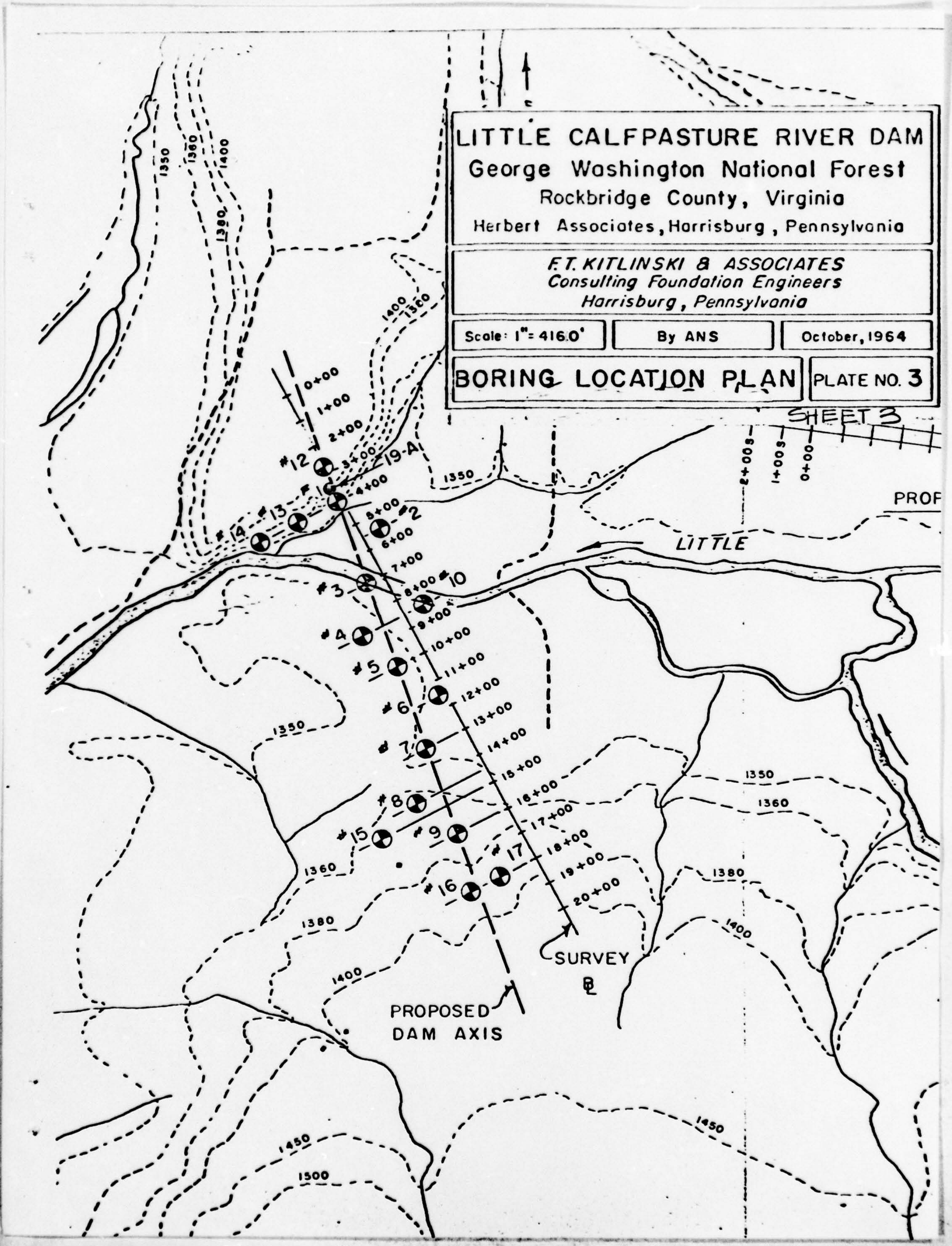
MAPS AND DRAWINGS

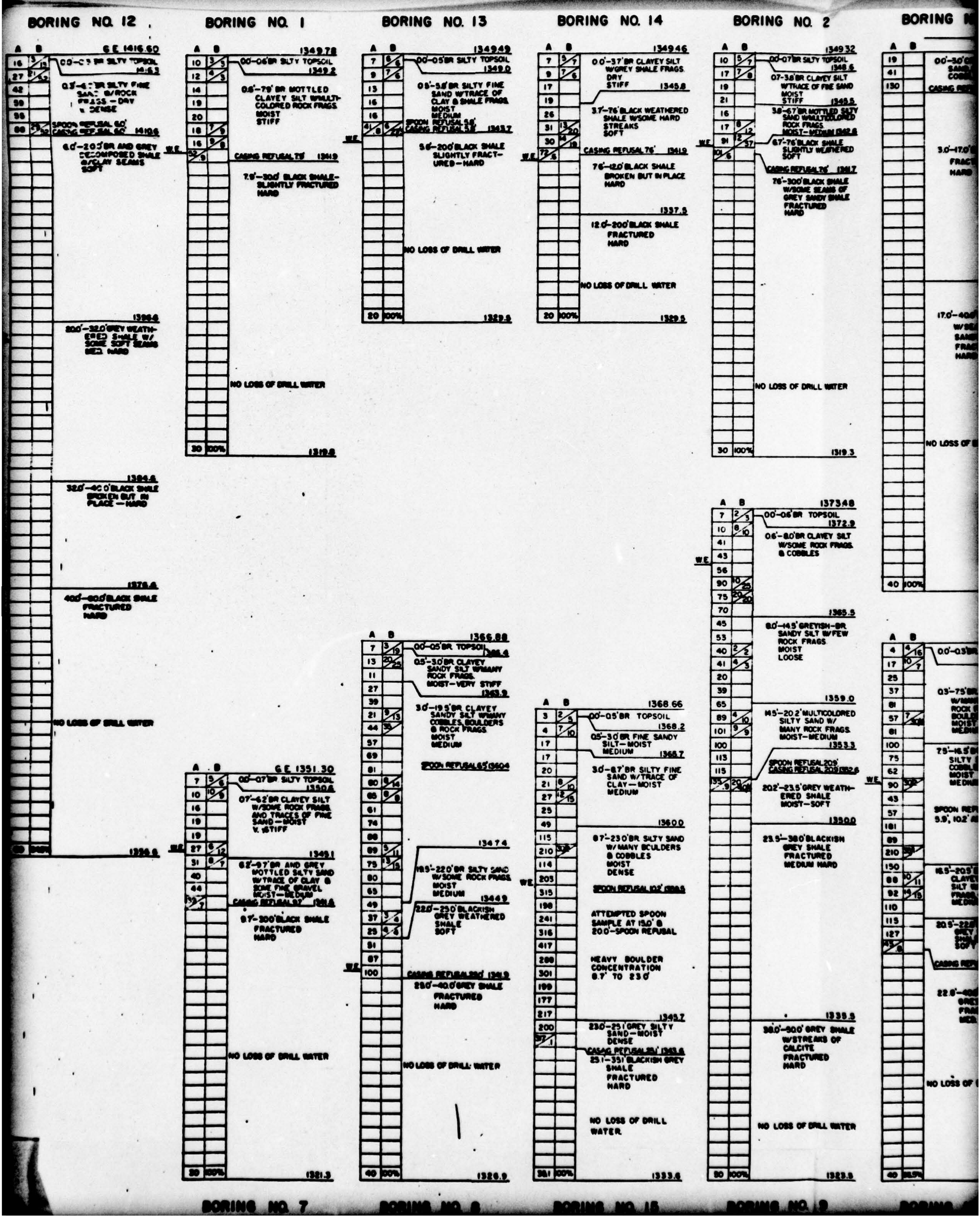


LOCATION MAP
SHEET 1
SCALE 1"=2000'±
GOSHEN, VA.
NW/4 LEXINGTON 15 QUADRANGLE
N3752.5—W7922.5/7.5
LAKE MERRIWEATHER
LAKE MERRIWEATHER DAM
LITTLE RIVER
CALFPASTURE RIVER
GOSHEN MAURY
Mountain
Knob
Camp Virginia
Little River Cem
Water Tank
Rock Hollow
Hickory Hollow
Chambers Hollow
Chambers Ridge
Round Knob
Kelso Spr
BM 1417
BM 1437
BM 1439
BM 1430
BM 1420
BM 1416
BM 1497
BM 1371
1369
1368
Goshen
Millboro
Millboro Springs
Goshen Pass
Brownsburg
Rockbridge Baths
ROCKBRIDGE
Virginia Military Institute
Washington & Lee University
LEXINGTON
BUENA VISTA



SHEET 2
River
Mountain
42
Calfpasture
THE KNOB ROAD
GOSHEN
Knob
McDONALD STORE
Camp Virginia
Brallon Mountain
Kelso Spring
Mountain
George Washington National Forest
North
Little
Rock Hollow
Hickory Hollow
Calfpasture River
PROPOSED DAM SITE
Little
Chambers Hollow
Goshen Pass
ROCKBRIDGE COUNTY
Forge Mountain
39
North River
Hogback Mountain
N
LITTLE CALFPASTURE RIVER DAM
George Washington National Forest
Rockbridge County, Virginia
Herbert Associates, Harrisburg, Pennsylvania
F.T. KITLINSKI & ASSOCIATES
Consulting Foundation Engineers
Harrisburg, Pennsylvania
Scale: 1" = 1.0 mi.
By ANS
October, 1964
SITE LOCATION PLAN
PLATE NO.


LITTLE CALFPASTURE RIVER DAM
George Washington National Forest
Rockbridge County, Virginia
Herbert Associates, Harrisburg, Pennsylvania
F.T. KITLINSKI & ASSOCIATES
Consulting Foundation Engineers
Harrisburg, Pennsylvania
Scale: 1" = 416.0'
By ANS
October, 1964
BORING LOCATION PLAN
PLATE NO. 3
SHEET 3
PROPOSED DAM AXIS
SURVEY ℄
LITTLE
PROF
#12
19-A
#13
#14
#2
#3
#10
#4
#5
#6
#7
#8
#15
#9
#17
#16
0+00
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
10+00
11+00
12+00
13+00
14+00
15+00
16+00
17+00
18+00
19+00
20+00
1350
1360
1380
1400
1450
1500



BORING NO. 12
G.E. 1416.60
0.0'-0.3' BR SILTY TOPSOIL
SPOON REFUSAL 6.0'
CASING REFUSAL 6.0' 1410.6
6.0'-20.0' BR AND GREY DECOMPOSED SHALE W/CLAY SEAMS SOFT
1396.6
20.0'-32.0' GREY WEATHERED SHALE W/ SOME SOFT SEAMS MED. HARD
1384.6
32.0'-40.0' BLACK SHALE BROKEN BUT IN PLACE - HARD
1376.6
40.0'-60.0' BLACK SHALE FRACTURED HARD
NO LOSS OF DRILL WATER
1356.6
BORING NO. 1
1349.78
0.0'-0.6' BR SILTY TOPSOIL
1349.2
0.6'-7.9' BR MOTTLED CLAYEY SILT W/MULTI-COLORED ROCK FRAGS. MOIST STIFF
CASING REFUSAL 7.9' 1341.9
7.9'-30.0' BLACK SHALE-SLIGHTLY FRACTURED HARD
NO LOSS OF DRILL WATER
30 100%
1319.8
BORING NO. 13
1349.49
0.0'-0.5' BR SILTY TOPSOIL
1349.0
0.5'-5.8' BR SILTY FINE SAND W/TRACE OF CLAY & SHALE FRAGS. MOIST MEDIUM
SPOON REFUSAL 5.8'
CASING REFUSAL 5.8' 1343.7
5.8'-20.0' BLACK SHALE SLIGHTLY FRACTURED-HARD
NO LOSS OF DRILL WATER
20 100%
1329.5
BORING NO. 14
1349.46
0.0'-3.7' BR CLAYEY SILT W/GREY SHALE FRAGS. DRY STIFF 1345.8
3.7'-7.6' BLACK WEATHERED SHALE W/SOME HARD STREAKS SOFT
CASING REFUSAL 7.6' 1341.9
7.6'-12.0' BLACK SHALE BROKEN BUT IN PLACE HARD
1337.5
12.0'-20.0' BLACK SHALE FRACTURED HARD
NO LOSS OF DRILL WATER
20 100%
1329.5
BORING NO. 2
1349.32
0.0'-0.7' BR SILTY TOPSOIL
1348.6
0.7'-3.8' BR CLAYEY SILT W/TRACE OF FINE SAND MOIST STIFF 1345.5
3.8'-6.7' BR MOTTLED SILTY SAND W/MULTICOLORED ROCK FRAGS. MOIST-MEDIUM 1342.6
6.7'-7.6' BLACK SHALE SLIGHTLY WEATHERED SOFT
CASING REFUSAL 7.6' 1341.7
7.6'-30.0' BLACK SHALE W/SOME SEAMS OF GREY SANDY SHALE FRACTURED HARD
NO LOSS OF DRILL WATER
30 100%
1319.3
BORING NO.
3.0'-17.0'
17.0'-40.0'
NO LOSS OF
40 100%
G.E. 1351.30
0.0'-0.7' BR SILTY TOPSOIL
1350.6
0.7'-6.2' BR CLAYEY SILT W/SOME ROCK FRAGS. AND TRACES OF FINE SAND-MOIST V. STIFF
1349.1
6.2'-9.7' BR AND GREY MOTTLED SILTY SAND W/TRACE OF CLAY & SOME FINE GRAVEL MOIST-MEDIUM
CASING REFUSAL 9.7' 1341.6
9.7'-30.0' BLACK SHALE FRACTURED HARD
NO LOSS OF DRILL WATER
30 100%
1321.3
BORING NO. 7
1366.88
0.0'-0.5' BR TOPSOIL
1366.4
0.5'-3.0' BR CLAYEY SANDY SILT W/MANY ROCK FRAGS. MOIST-VERY STIFF
1363.9
3.0'-19.5' BR CLAYEY SANDY SILT W/MANY COBBLES, BOULDERS & ROCK FRAGS. MOIST MEDIUM
SPOON REFUSAL 6.5' 1360.4
1347.4
19.5'-22.0' BR SILTY SAND W/SOME ROCK FRAGS. MOIST MEDIUM
1344.9
22.0'-23.0' BLACKISH GREY WEATHERED SHALE SOFT
CASING REFUSAL 23.0' 1343.9
23.0'-40.0' GREY SHALE FRACTURED HARD
NO LOSS OF DRILL WATER
40 100%
1326.9
BORING NO. 8
1368.66
0.0'-0.5' BR TOPSOIL
1368.2
0.5'-3.0' BR FINE SANDY SILT-MOIST MEDIUM
1365.7
3.0'-8.7' BR SILTY FINE SAND W/TRACE OF CLAY-MOIST MEDIUM
1360.0
8.7'-23.0' BR SILTY SAND W/MANY BOULDERS & COBBLES MOIST DENSE
SPOON REFUSAL 10.2' 1358.5
ATTEMPTED SPOON SAMPLE AT 13.0' & 20.0'-SPOON REFUSAL
HEAVY BOULDER CONCENTRATION 8.7' TO 23.0'
1345.7
23.0'-25.1' GREY SILTY SAND-MOIST DENSE
CASING REFUSAL 25.1' 1343.6
25.1'-35.1' BLACKISH GREY SHALE FRACTURED HARD
NO LOSS OF DRILL WATER
35.1 100%
1333.6
BORING NO. 15
1373.48
0.0'-0.6' BR TOPSOIL
1372.9
0.6'-8.0' BR CLAYEY SILT W/SOME ROCK FRAGS. & COBBLES
1365.5
8.0'-14.5' GREYISH-BR SANDY SILT W/FEW ROCK FRAGS. MOIST LOOSE
1359.0
14.5'-20.2' MULTICOLORED SILTY SAND W/ MANY ROCK FRAGS. MOIST-MEDIUM
1353.3
SPOON REFUSAL 20.9'
CASING REFUSAL 20.9' 1352.6
20.2'-23.5' GREY WEATHERED SHALE MOIST-SOFT
1350.0
23.5'-38.0' BLACKISH GREY SHALE FRACTURED MEDIUM HARD
1335.5
38.0'-50.0' GREY SHALE W/STREAKS OF CALCITE FRACTURED HARD
NO LOSS OF DRILL WATER
50 100%
1323.5
BORING NO. 9
0.0'-0.5' BR
0.5'-7.5' BR
7.5'-16.5'
SPOON REFUSAL 5.9', 10.2'
16.5'-20.9'
20.9'-22.8'
CASING REFUSAL
22.8'-40.0'
NO LOSS OF
40 96.5%
BORING NO.

BORING NO. 3

W.E. 1343.06

A	B
19	
41	
130	
40	100%

1342.06

0.0'–3.0' CREEK BED SAND, GRAVEL AND COBBLES

CASING REFUSAL 3.0' 1339.1

3.0'–17.0' BLACK SHALE FRACTURED HARD

1325.1

17.0'–40.0' BLACK SHALE W/SEAMS OF GREY SANDY SHALE FRACTURED HARD

NO LOSS OF DRILL WATER

1302.1

BORING NO. 10

A	B
3	1/2
5	3/3
7	
11	
25	
51	4/9
34	15/21
37	
52	
75/9	
20	100%

1349.96

0.0'–0.9' BR TOPSOIL 1349.1

0.9'–4.5' BR CLAYEY SILT W/SOME ROCK FRAGS MOIST SOFT

1345.5

4.5'–9.9' BR SILTY SAND W/ROCK FRAGS AND COBBLES MOIST MEDIUM

W.E.

CASING REFUSAL 9.9' 1340.1

9.9'–20.0' GREY SHALE W/STREAKS OF CALCITE FRACTURED HARD

NO LOSS OF DRILL WATER

1330.0

BORING NO. 4

A	B
6	3/5
9	6/5
12	
14	
17	
8	1/1
9	2/2
12	
73/4	
30	100%

1349.62

0.0'–0.6' BR SILTY TOPSOIL 1349.0

0.6'–4.9' BR CLAYEY SILT W/TRACES OF FINE SAND MOIST STIFF 1344.7

4.9'–5.5' BR SILTY FINE TO MED SAND—WET V. LOOSE 1344.1

W.E.

5.5'–8.4' GREY SILTY FINE TO MED SAND W/SOME FINE GRAVEL WET—V. LOOSE

CASING REFUSAL 8.4' 1341.2

8.4'–30.0' GREY SANDY SHALE—SLIGHTLY FRACTURED HARD

1319.6

BORING NO. 5

A	B
10	3/5
26	5/6
27	
28	
29	
31	3/2
25	2/3
47	
33	
21	
20	7/9
57	15/25
75/3	
30	88%

1350.55

0.0'–0.8' BR TOPSOIL 1349.8

0.8'–4.0' BR CLAYEY SILT W/SOME ROCK FRAGS. MOIST—STIFF 1346.6

4.0'–6.5' GREY MED SAND W/SOME ROCK FRAGS. MOIST—LOOSE 1344.1

W.E.

6.5'–9.5' BR SANDY SILT W/TRACE OF CLAY & SOME ROCK FRAGS. MOIST—MEDIUM 1341.1

9.5'–11.0' GREY SILT MOIST—SOFT 1339.6

11.0'–12.3' BLACKISH GREY BADLY WEATHERED SHALE—MOIST—SOFT

CASING REFUSAL 12.3' 1338.3

12.3'–30.0' GREY SHALE W/STREAKS OF CALCITE FRACTURED HARD

NO LOSS OF DRILL WATER

1320.6

BORING NO. 6

A	B
12	5/7
9	7/9
16	
13	
19	
10	2/2
7	3/4
7	
34	
84/5	
30	97%

G.E. 1350.84

0.0'–0.7' BR SILTY TOPSOIL 1350.1

0.7'–4.7' BR CLAYEY SILT W/TRACE OF FINE SAND—MOIST—STIFF

1346.1

W.E.

4.7'–8.8' BR SILTY MED TO FINE SAND W/SOME FINE GRAVEL WET LOOSE 1342.0

8.8'–9.5' BLACK SHALE WEATHERED—SOFT

CASING REFUSAL 9.5' 1341.3

9.5'–30.0' BLACK SHALE FRACTURED HARD

NO LOSS OF DRILL WATER

1320.8

BORING NO. 17

A	B
3	2/3
8	4/5
9	
11	
15	
17	2/3
19	3/4
37	
115	
81	
45	15/31
94	
213	
199	
315	
419	
214	
177	
233	
179	
211	
155	5/7
160	11/14
179	
200	
203	
250	10/31
60	88.7%

G.E. 1391.53

0.0'–0.6' BR. TOPSOIL

1390.9

0.6'–7.5' BR MOTTLED SANDY SILT W/FEW ROCK FRAGS. MOIST LOOSE

1384.0

7.5'–20.5' BR MOTTLED SANDY SILT W/HEAVY CONCENTRATION OF COBBLES & BOULDERS MOIST DENSE

W.E.

SPOON REFUSAL 11.0' 1380.5

DRILLED BOULDERS FROM 8.0' TO 20.0'

1371.0

20.5'–25.0' BR MOTTLED SILTY CLAY W/FEW SHALE FRAGS. MOIST STIFF

1366.5

25.0'–27.0' BLACKISH GREY WEATHERED SHALE—SOFT.

CASING REFUSAL 27.0' 1364.5

27.0'–60.0' BLACKISH GREY SHALE FRACTURED HARD

ATTEMPTED SPOON SAMPLE AT 15.0' & 27.0'—SPOON REFUSAL

1331.5

BORING NO. 17

BORING NO. 16

A	B
4	4/16
17	10/7
25	
37	
81	
57	7/30
81	
100	
75	
62	
90	30/2
43	
57	
181	
89	
210	30
150	
88	10/11
92	14/15
110	
115	
127	
45/8	
40	95.5%

1394.52

0.0'–0.3' BR TOPSOIL 1394.2

0.3'–7.5' BR SANDY SILT W/MANY COBBLES, ROCK FRAGS. & BOULDERS MOIST MEDIUM

1387.0

7.5'–16.5' BR. MOTTLED SILTY SAND, GRAVELS, COBBLES & BOULDERS MOIST MEDIUM

W.E.

SPOON REFUSALS AT 5.9', 10.2' AND 15.1'

1378.0

16.5'–20.5' BR MOTTLED CLAYEY SANDY SILT W/FEW ROCK FRAGS.—MOIST MEDIUM

1374.0

20.5'–22.8' BLACKISH GREY WEATHERED SHALE—MOIST SOFT

CASING REFUSAL 22.8' 1371.7

22.8'–40.0' BLACKISH GREY SHALE FRACTURED MED. HARD

NO LOSS OF DRILL WATER

1354.5

BORING NO. 16

2

NOTES:

BORINGS ARE SHOWN FROM WEST TO EAST ALONG DAM AXIS.

COLUMN "A" DENOTES BLOWS ON CASING PER FOOT OF PENETRATION OR LOWER LIMIT OF CORE RUN.

COLUMN "B" DENOTES BLOWS ON SAMPLING SPOON PER FOOT OF PENETRATION (EXCEPT AS NOTED) OR PERCENT OF CORE RECOVERY.

INDICATES BLOWS FOR .5' (EXCEPT AS NOTED)

VERTICAL SCALE 1" = 40'

SIZE OF CASING — 4 INCHES
WT. OF HAMMER ON CASING — 300 POUNDS
DROP OF HAMMER ON CASING — 18 INCHES
SIZE OF SPOON — 2 INCHES O.D.
WT. OF HAMMER ON SPOON — 140 POUNDS
DROP OF HAMMER ON SPOON — 30 INCHES
SIZE OF CORE NXM — 2-1/8 INCHES

G.E. — INDICATES GROUND ELEVATION
W.E. — INDICATES WATER ELEVATION

SEE PARAGRAPH S.P. 11 IN THE SPECIAL PROVISIONS ENTITLED FOUNDATION AND MATERIAL INVESTIGATION.

SHEET 4

NATIONAL CAPITAL AREA COUNCIL
BOY SCOUTS OF AMERICA

IMPOUNDING DAM
GOSHEN, ROCKBRIDGE COUNTY, VIRGINIA

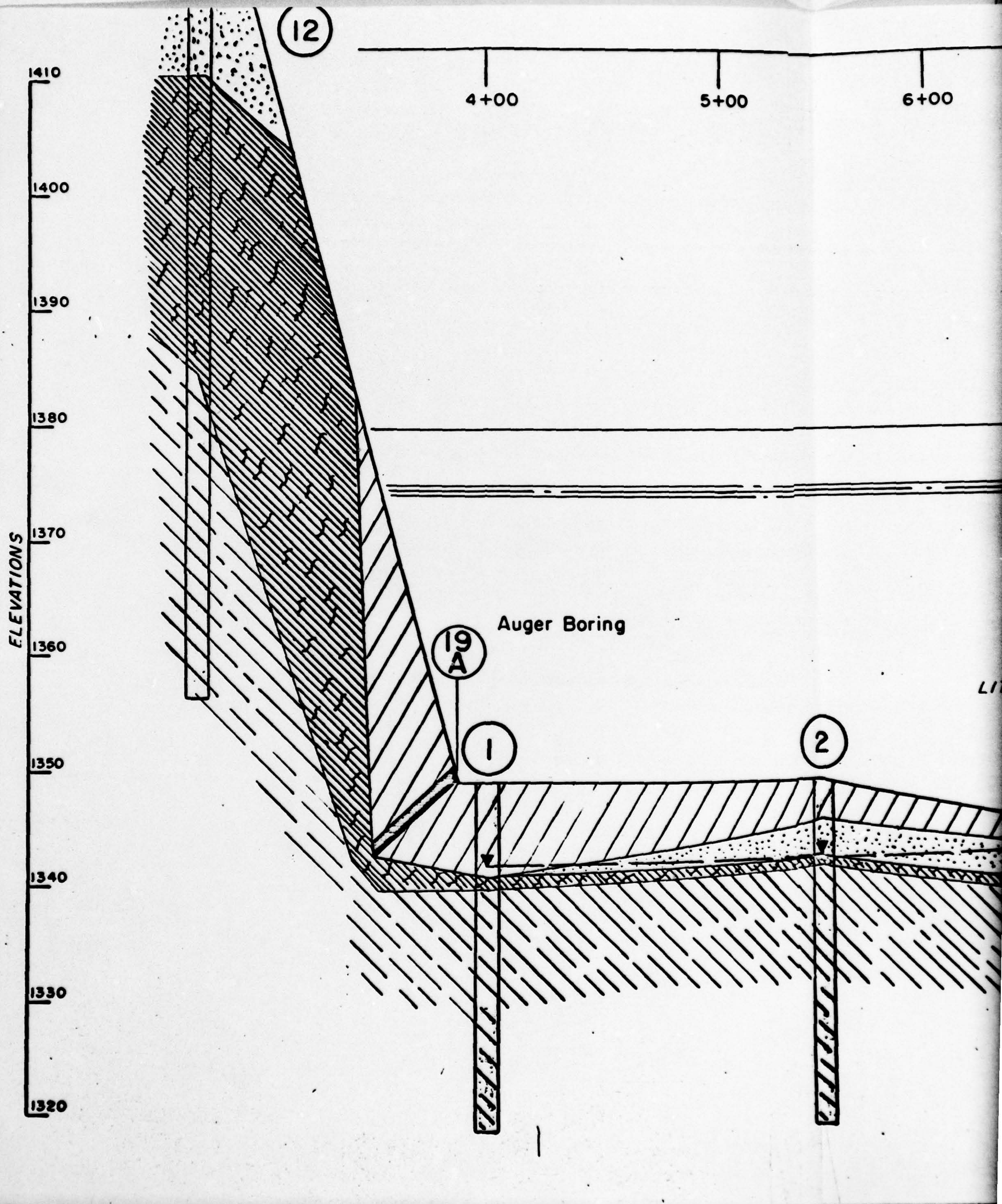
CORE BORINGS

HERBERT ASSOCIATES, Inc.
CONSULTING ENGINEERS

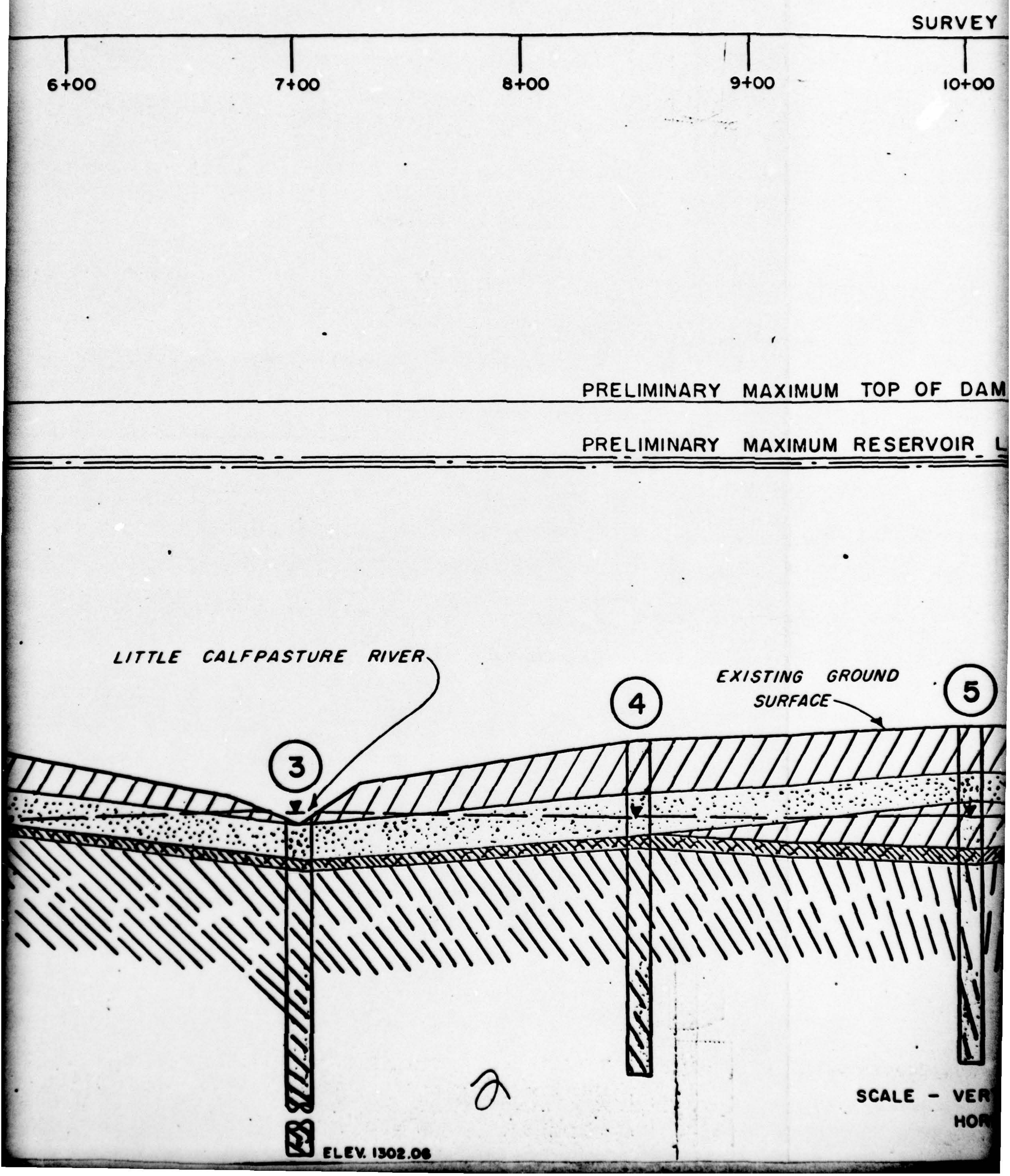
R.R.
D.S.

1" = 40'

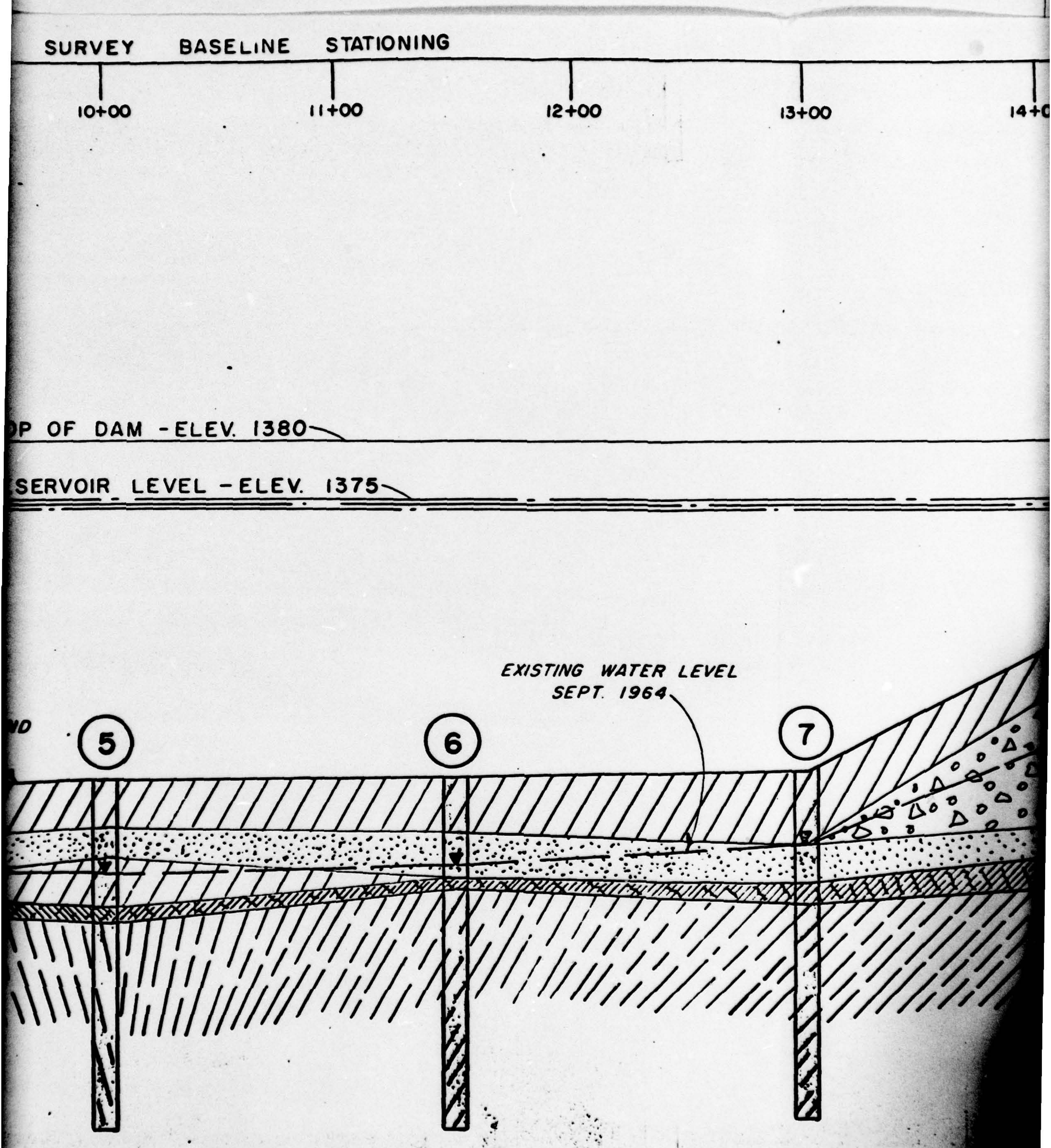
9



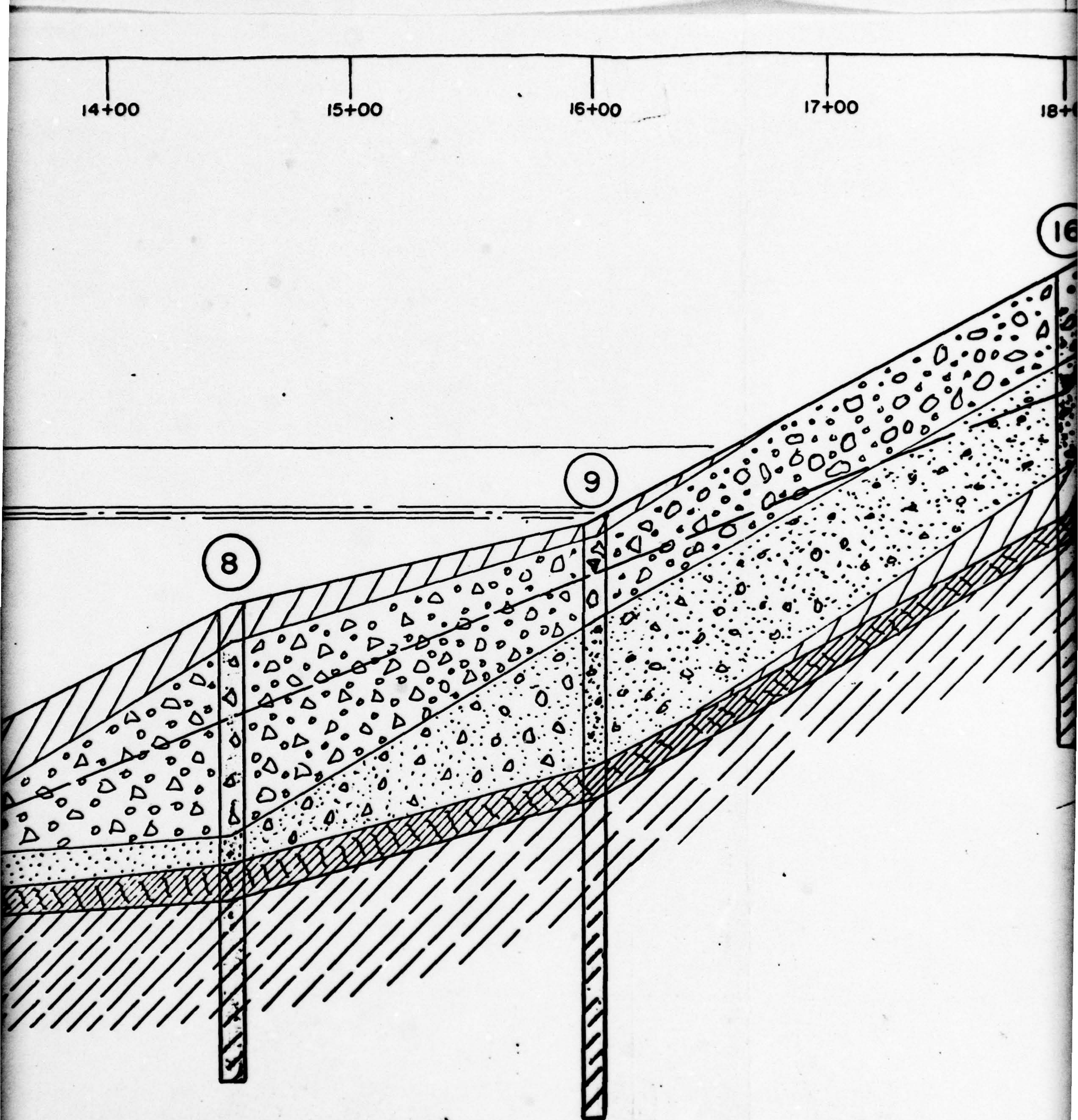
ELEVATIONS
1410
1400
1390
1380
1370
1360
1350
1340
1330
1320
4+00
5+00
6+00
12
19
A
Auger Boring
1
2



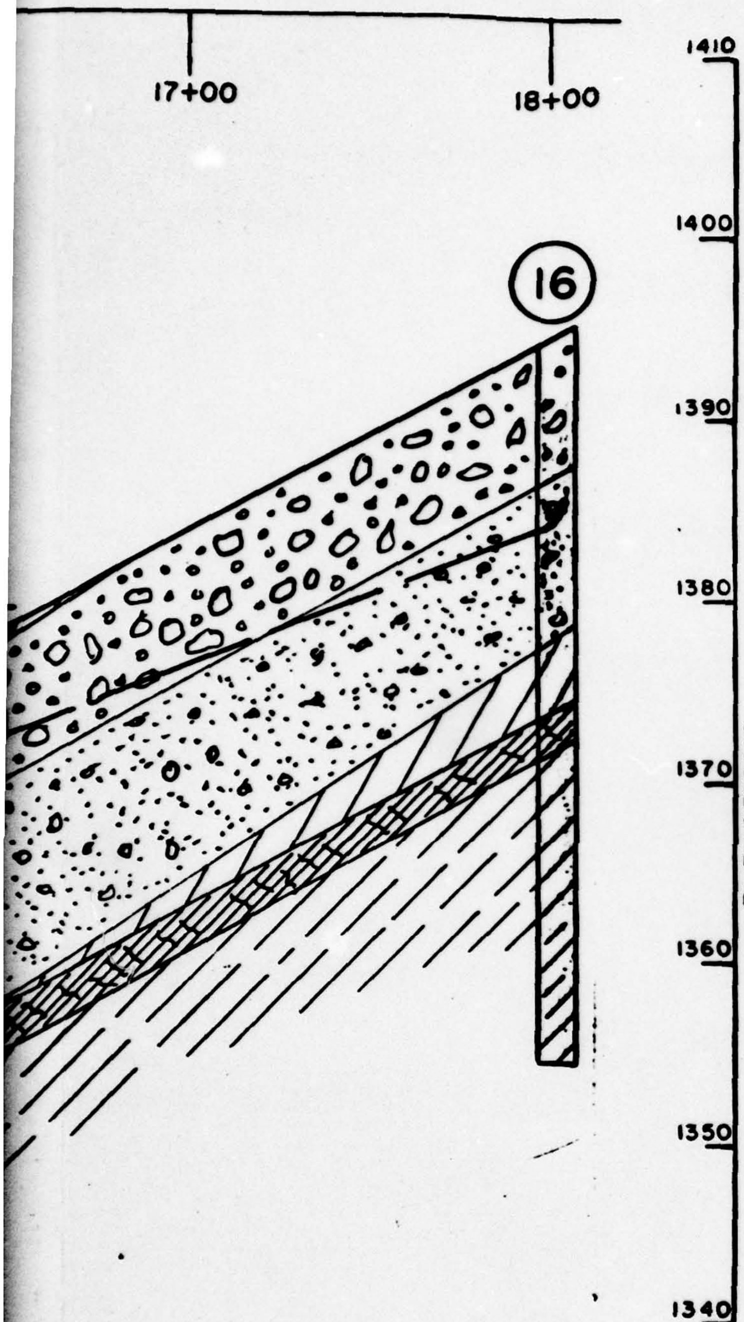
SURVEY
6+00
7+00
8+00
9+00
10+00
PRELIMINARY MAXIMUM TOP OF DAM
PRELIMINARY MAXIMUM RESERVOIR L
LITTLE CALFPASTURE RIVER
EXISTING GROUND SURFACE
3
4
5
ELEV. 1302.06
SCALE - VER
HOR


SURVEY BASELINE STATIONING
10+00
11+00
12+00
13+00
14+0
OP OF DAM - ELEV. 1380
SERVOIR LEVEL - ELEV. 1375
EXISTING WATER LEVEL
SEPT. 1964
ND
5
6
7
ALE - VERTICAL: 1" = 10'
HORIZONTAL: 1" = 50'

3



4



— LEGEND —

ML-CL - SILTS & CLAYS

SM - SILTY SANDS
SC - CLAYEY SANDS

BOULDERS & COBBLES

SP - SILTY SANDS & COBBLES

WEATHERED SHALE

BLACK MILLBORO SHALE

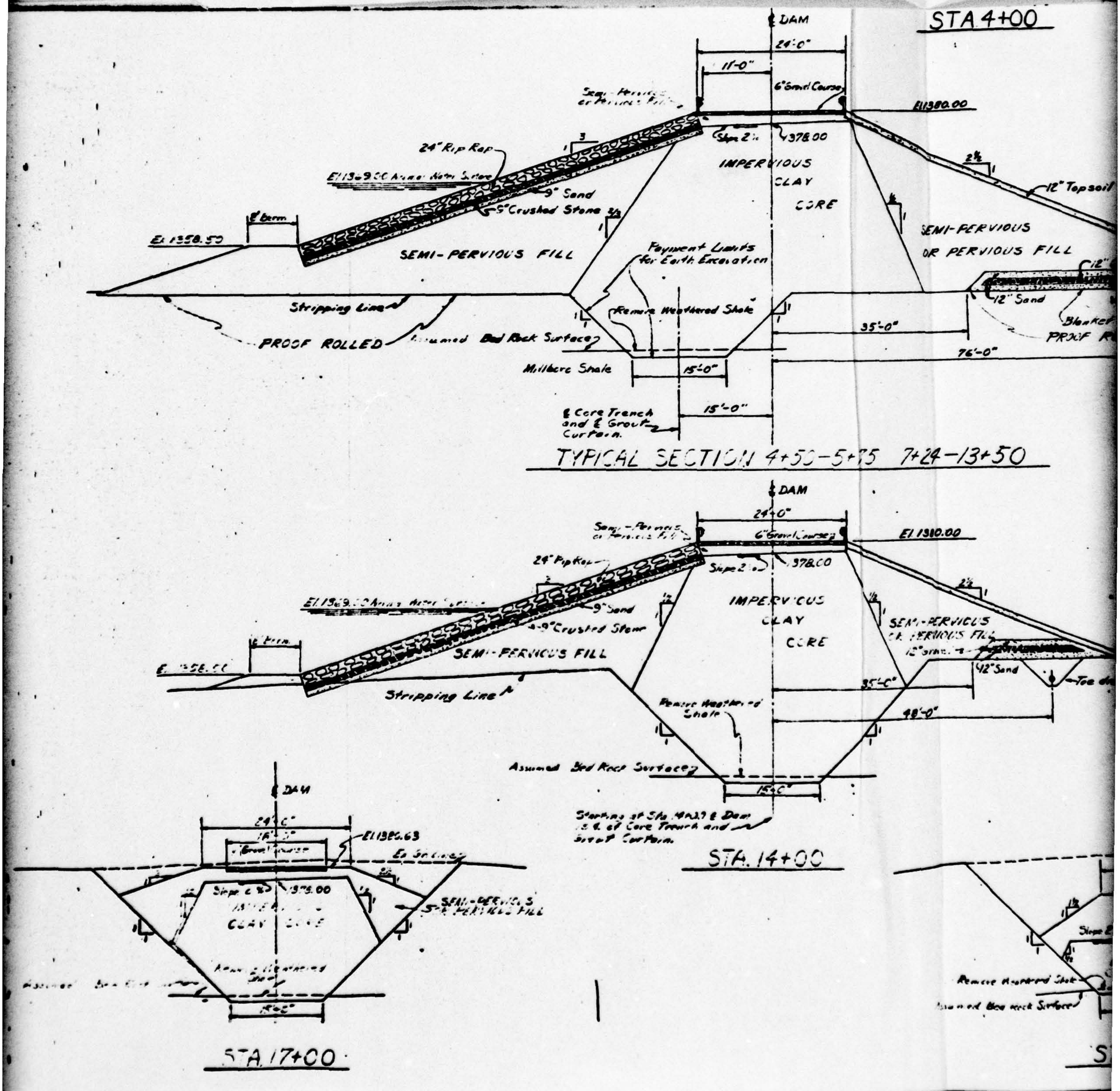
(3) BORING NUMBER

NOTE - SEE PLATE NO. 3 FOR BORING LAYOUT

5

SHEET 5

LITTLE CALFPASTURE RIVER DAM George Washington National Forest Rockbridge County, Virginia Herbert Associates, Harrisburg, Pennsylvania		
F.T. KITLINSKI & ASSOCIATES *Consulting Foundation Engineers* *Harrisburg, Pennsylvania*		
Scale: As Noted	By ANS	October, 1964
GEOLOGIC PROFILE		PLATE NO. 4


STA 4+00
℄ DAM
24'-0"
11'-0"
6" Gravel Course
El 1380.00
1378.00
24" Rip Rap
IMPERVIOUS
CLAY
CORE
12" Topsoil
9" Sand
5" Crushed Stone
El 1358.50
SEMI-PERVIOUS FILL
SEMI-PERVIOUS
OR PERVIOUS FILL
Payment Limits
for Earth Excavation
Stripping Line
Remove Weathered Shale
12" Sand
35'-0"
PROOF ROLLED
Assumed Bed Rock Surface
76'-0"
Millbore Shale
15'-0"
℄ Core Trench
and ℄ Grout
Curtain
TYPICAL SECTION 4+50-5+75 7+24-13+50
℄ DAM
24'-0"
6" Gravel Course
El 1380.00
24" Rip Rap
1378.00
9" Sand
9" Crushed Stone
IMPERVIOUS
CLAY
CORE
SEMI-PERVIOUS FILL
42" Sand
35'-0"
Stripping Line
48'-0"
Assumed Bed Rock Surface
STA. 14+00
℄ DAM
24'-0"
El 1380.63
STA. 17+00
Remove Weathered Shale
Assumed Bed Rock Surface

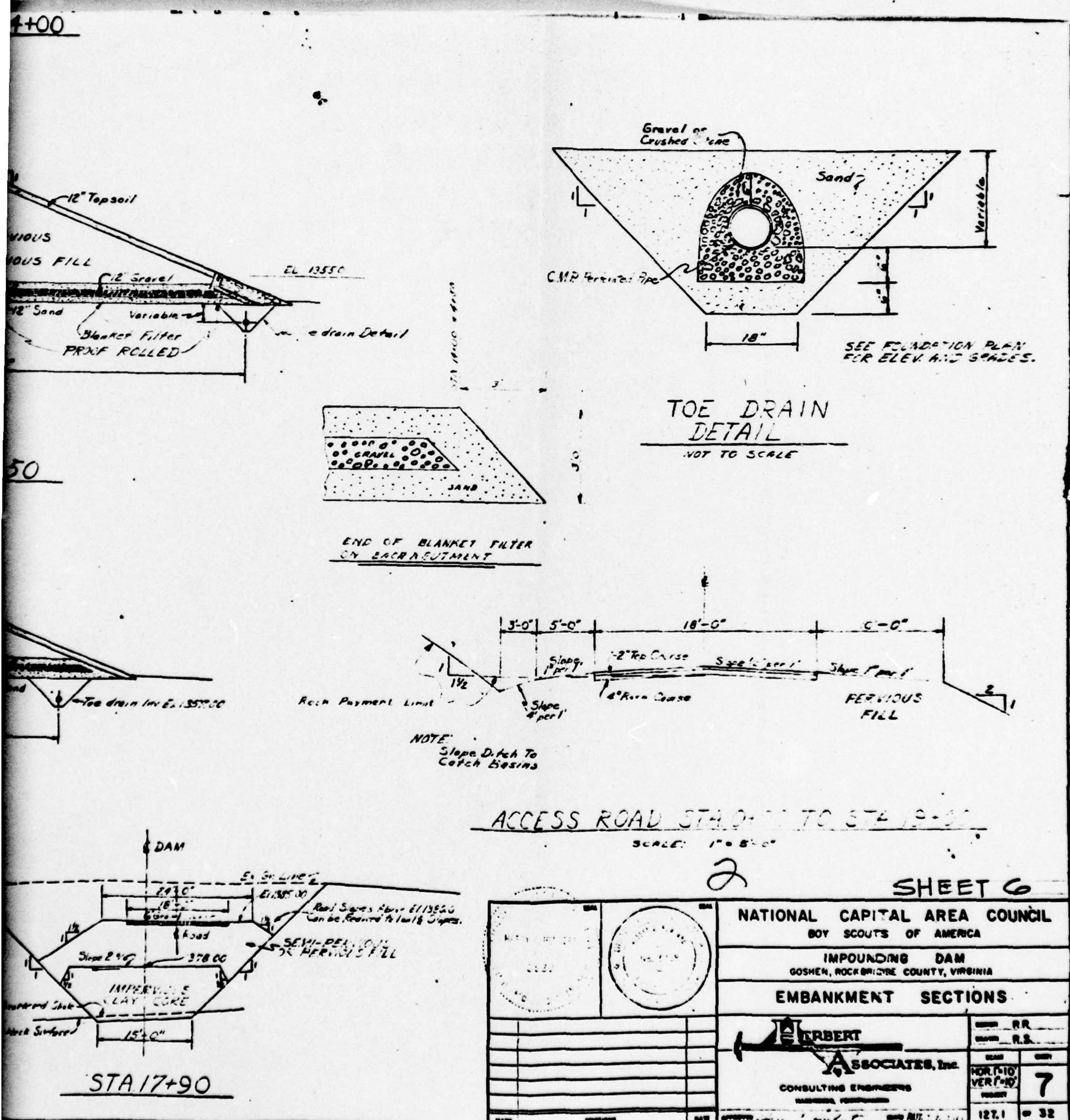

4+00
12" Topsoil
12" Gravel
EL 1355.0
12" Sand
Variable
Blanket Filter
PROOF ROLLED
Gravel or
Crushed Stone
Sand
Variable
C.M.P. Perforated Pipe
18"
SEE FOUNDATION PLAN
FOR ELEV. AND GRADES.
TOE DRAIN
DETAIL
NOT TO SCALE
GRAVEL
SAND
END OF BLANKET FILTER
ON EACH ABUTMENT
50
Toe drain Inv El 1351.00
3'-0"
5'-0"
18'-0"
Slope 1" per 1'
2" Top Course
4" Rock Course
Rock Payment Limit
PERVIOUS
FILL
NOTE:
Slope Ditch To
Catch Basins
ACCESS ROAD
SCALE: 1" = 8'-0"
DAM
Ex Gr Line
Road
SEMI-PERVIOUS
OR PERVIOUS FILL
IMPERVIOUS
CLAY CORE
15'-0"
STA 17+90
2
SHEET 6
NATIONAL CAPITAL AREA COUNCIL
BOY SCOUTS OF AMERICA
IMPOUNDING DAM
GOSHEN, ROCKBRIDGE COUNTY, VIRGINIA
EMBANKMENT SECTIONS
HERBERT
ASSOCIATES, Inc.
CONSULTING ENGINEERS
R.R.
R.S.
HOR. 1"=10'
VER. 1"=10'
7
127.1
32

TABLE II

SUMMARY OF LABORATORY

LITTLE CALFPASTURE RIVER

Rockbridge County, Virgini

BORING NO.	DEPTH	LL	PL	PI	% FINER #4	% FINER #200	NATURAL MOIST. %	STD. COMPAC	
								OPT. MOIST. %	MA DE
						DAM	FOUNDATION		
1	5.0' - 7.0'	45	28	17	100	88	35.3	—	
2	5.0' - 7.0'	24	20	4	73	34	16.1	—	
4	0.0' - 2.0'	34	21	13	100	79	17.0	—	
5	5.0' - 7.0'	24	18	6	76	46	25.6	—	
5	10.0' - 12.0'	34	28	6	97	82	26.2	—	
7	0.0' - 2.0'	36	21	15	100	85	17.4	—	
7	5.0' - 7.0'	—	NP	—	84	49	16.6	—	
9	5.0' - 7.0'	—	NP	—	71	27	8.3	—	
9	10.0' - 12.0'	—	NP	—	93	34	18.1	—	
12	5.0' - 6.0'	46	32	16	100	46	18.8	—	
17	5.0' - 6.0'	23	18	5	100	54	16.6	—	
17	21.0' - 23.0'	35	23	12	98	76	27.6	—	
						BORROW		AREA	
2	4.0' - 5.0'	41	20	21	100	90	19.8	—	
3	5.0' - 7.0'	46	21	25	100	95	21.8	—	
6	8.0' - 10.0'	22	17	5	82	48	14.1	—	
8	6.0' - 7.0'	37	21	16	96	63	14.8	—	
10	7.0' - 10.0'	28	18	10	76	28	9.9	—	
13	0.0' - 5.0'	41	23	18	100	94	21.1	—	
13	6.0' - 7.0'	43	23	20	88	61	19.0	—	
21	4.0' - 5.0'	23	15	8	100	57	9.2	—	
21	6.0' - 7.0'	30	15	15	66	42	10.6	—	
21	8.0' - 8.5'	26	16	10	87	34	9.6	—	
5	4.0' - 5.0'	52	30	22	100	86	21.6	—	
5	5.0' - 10.0'	31	18	13	99	63	18.8	—	
7	0.0' - 6.0'	38	18	20	100	88	23.4	—	
9	0.0' - 6.0'	36	19	17	99	78	17.5	—	
12	0.0' - 9.0'	30	18	12	89	56	14.4	16.6	
16	6.0' - 10.0'	29	18	11	99	65	20.2	17.6	
18	0.4' - 5.5'	51	24	27	96	74	17.0	19.8	
18	5.5' - 10.0'	29	20	9	86	38	9.5	—	
19	0.0' - 6.0'	31	17	14	99	72	32.4	—	
20	0.0' - 6.0'	48	28	20	96	78	17.5	23.0	

F. T. KITLINSKI & ASSOCIATES
Harrisburg, Pennsylvania
OCTOBER, 1964

TABLE II

...ARY OF LABORATORY TESTS

...LE CALFPASTURE RIVER DAM

Rockbridge County, Virginia

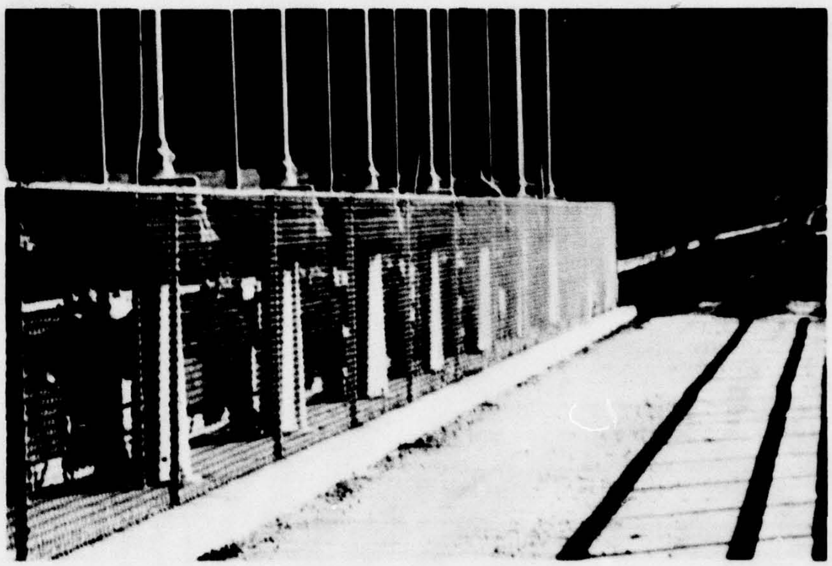
...INER ...00	NATURAL MOIST. %	STD. COMPACTION		UNIFIED SOIL CLASS.	COEFF. OF PERMEABILITY cm/sec @ 20° c	DIRECT SHEAR	
		OPT. MOIST. %	MAX. DRY DENSITY			COHESION PSF	FRICTION °
DAM	FOUNDATION						
8	35.3	—	—	ML	—	—	—
4	16.1	—	—	SC	—	—	—
9	17.0	—	—	CL	—	—	—
6	25.6	—	—	SM-SC	—	—	—
2	26.2	—	—	ML	—	—	—
5	17.4	—	—	CL	—	—	—
9	16.6	—	—	SM	—	—	—
7	8.3	—	—	SP	—	—	—
4	18.1	—	—	SP	—	—	—
6	18.8	—	—	SC	—	—	—
4	16.6	—	—	ML	—	—	—
6	27.6	—	—	CL	—	—	—
BORROW	AREA						
0	19.8	—	—	CL	—	—	—
5	21.8	—	—	CL	—	—	—
8	14.1	—	—	SM-SC	—	—	—
3	14.8	—	—	CL	—	—	—
8	9.9	—	—	SC	—	—	—
4	21.1	—	—	CL	—	—	—
1	19.0	—	—	CL	—	—	—
7	9.2	—	—	CL	—	—	—
2	10.6	—	—	SC	—	—	—
4	9.6	—	—	SC	—	—	—
6	21.6	—	—	MH	—	—	—
3	18.8	—	—	CL	—	—	—
8	23.4	—	—	CL	—	—	—
8	17.5	—	—	CL	—	—	—
6	14.4	16.6	110.8	SC	7.8×10^{-8}	576	28
5	20.2	17.6	110.2	CL	2.9×10^{-9}	1070	17
4	17.0	19.8	100.4	CH	1.3×10^{-7}	936	23.5
8	9.5	—	—	SC	—	—	—
2	32.4	—	—	CL	—	—	—
8	17.5	23.0	97.7	ML	8.7×10^{-7}	965	25

SHEET 7

2

APPENDIX II

PHOTOGRAPHS



VIEW OF GATE MOTOR ATOP SPILLWAY BRIDGE — Photo #1

VIEW OF TYPICAL CREST GATE (GATES IN DOWN POSITION) (10 GATES TOTAL) — Photo #2

UPSTREAM VIEW OF GATES
(Control Building in Background)
Photo #3

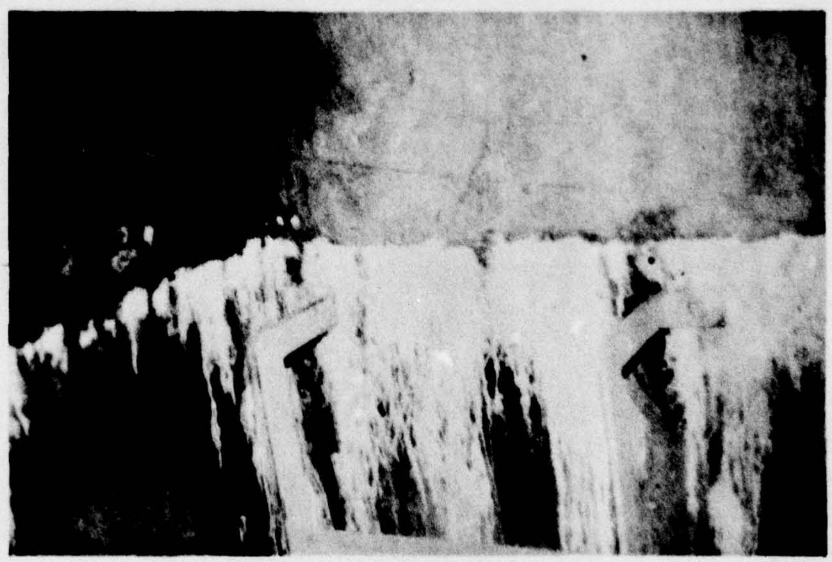
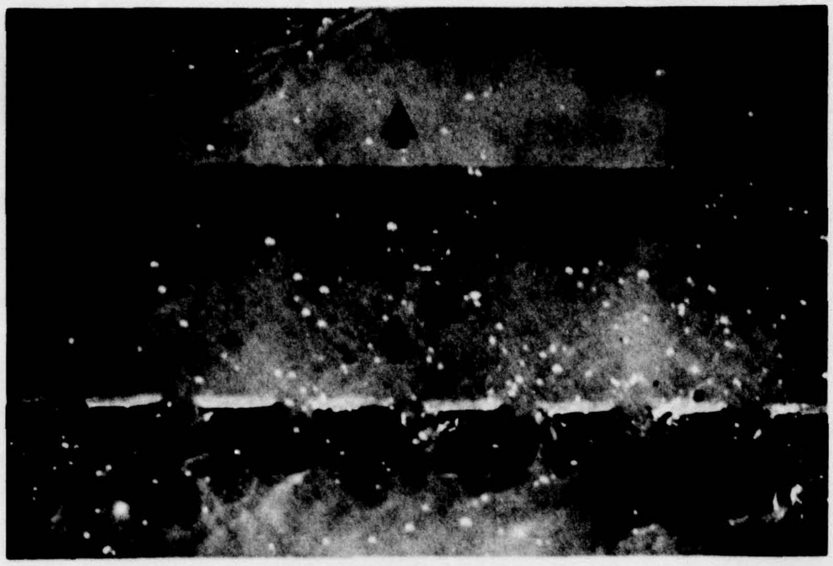
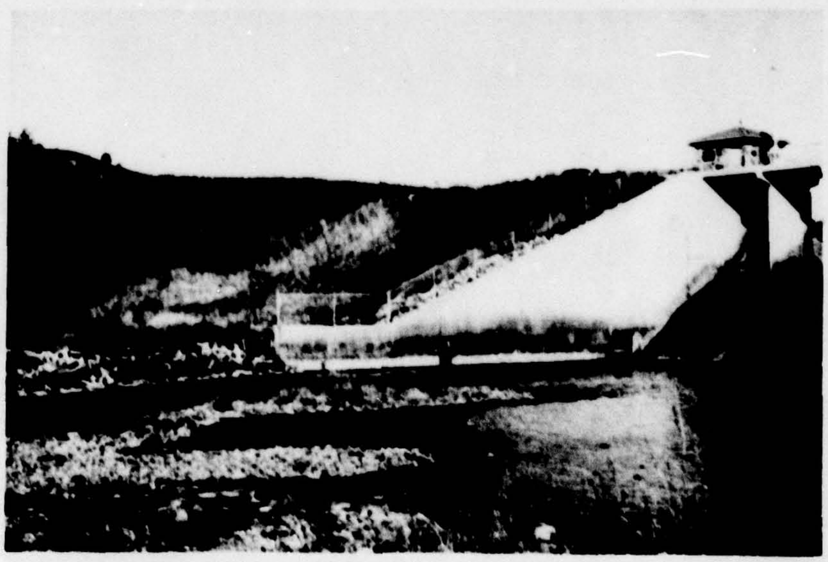


PHOTO SHOWING BLEEDING IN
SECOND LEVEL OUTLET CHAMBER
Photo #4

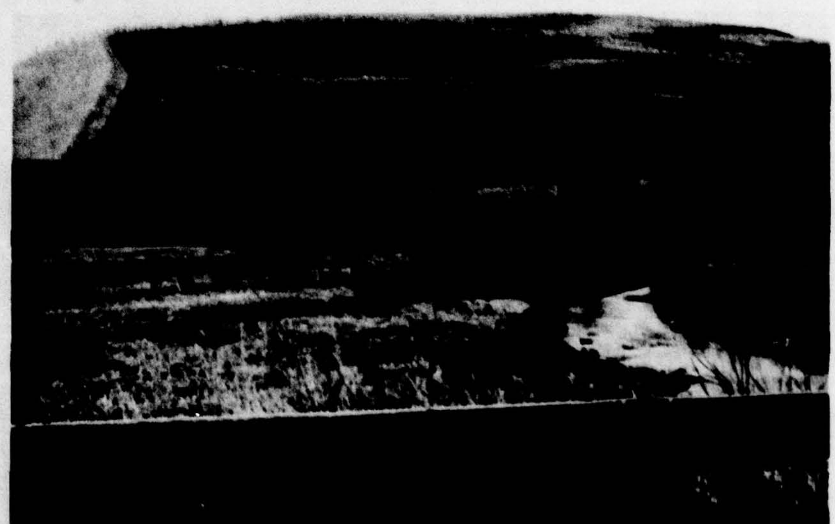


VIEW OF BUCKET AND TEETH AT TOE OF SPILLWAY (NOTE STILLING EFFECT ON WATER)
Photo #5



VIEW OF TYPICAL OUTLET WING WALL AT LOWER EDGE OF SPILLWAY
Photo #6

VIEW OF DOWNSTREAM CHANNEL AND FLOODPLAIN
FROM ℄ OF SPILLWAY TOWARD THE WEST
Photo #7



VIEW OF DOWNSTREAM CHANNEL AND FLOODPLAIN
FROM ℄ OF SPILLWAY TOWARD THE EAST
Photo #8

APPENDIX III

FIELD OBSERVATIONS

FIELD OBSERVATIONS

Name of Dam: Goshen
County: Rockbridge
State: Virginia
Coordinates: Lat. 37° - 57.57' Long 79° - 27.5'
Date of Inspection: December 14, 1978
Weather: Partly cloudy, temperature 35°
Pool Elevation at Time of Inspection: 1359.7
Tailwater at Time of Inspection: 1342.5
Inspection Personnel:

Schnabel Engineering Associates, P. C.
Ray E. Martin, P.E.
Stephen G. Werner (recorder)

J. K. Timmons and Associates, Inc.
Robert G. Roop, P.E.
William A. Johns (recorder)

State Water Control Board
John Hyden

1 Embankment:

1.1 Surface Cracks: The slopes, crest, emergency spillway, and abutment contacts were inspected and no cracks were noted. The downstream slope of the embankment was covered with 3 to 4 ft high, light vegetation, making observations difficult. Small (1 to 2" diameter) trees were growing along both sides of the spillway.

1.2 Unusual Movement: No unusual movements were noted on the dam or downstream beyond the embankment toe.

1.3 Sloughing or Erosion: No sloughing or erosion was noted.

1.4 Alignment: The vertical and horizontal alignment of the dam was visually observed to be in accordance with "as built" drawings.

1.5 Riprap: Showed no washed-out areas and appeared to be in good condition.

1.6 Junctions: Conditions appear good at the junction of the embankment and the abutments. Gentle slopes comprise the east abutment and includes scattered outcrops of dark gray fissile shale. Numerous outcrops of the same shale occur within the steep slope forming the west abutment. A steep drainage draw (dry) extends down the slope in line with the downstream toe of the dam. Along the middle of the west abutment hill, bedrock strikes about N58E to N76E. Bedrock dips 34° SE at the south end and increases to 48° SE along the middle of the hill. Local steepening of beds to essentially vertical was also observed. Scattered joints were present and included a rectangular set, ie., N23W, 90 and N65E, 45NW. Also measured the following joint: N5E, 83SW.

1.7 Seepage: No wet spots or seepage were noted on the embankment. A slight seep was observed flowing from riprap at the downsteam end of the west spillway wing wall. This seepage was below the level of the toe drain outfall and is believed to be toe drain seepage which is leaking from the drain. This seep does not appear to present any problem to the normal function of this dam.

1.8 Staff Gage: Gage is located in concrete surface of spillway and is in excellent condition.

1.9 Drains: The embankment contains a downstream toe drain which extends to about El 1350 along the north and south abutments. The drain contains a 6-inch diameter perforated CMP and is connected to a drainage blanket and filter extending upstream to within 35 ft of the dam centerline. Discharge outlets were noted on the wing walls of the spillway. A flap valve is provided on the drain existing from the west spillway wing wall. The flap valve is missing from the east wing wall drain. Slight seepage was flowing from both drain exits.

2 . Outlet Works:

2.1 Outlet Conduit: Outlet conduit in excellent condition.

2.2 Intake Structure: In good condition. Some construction joints in intake tower are bleeding.

2.3 Outlet Structure: Stilling basin in excellent condition.

2.4 Outlet Channel: In good condition.

2.5 Emergency Gate: None.

3 Gated Spillway:

3.1 Concrete Sill: In excellent condition.

3.2 Approach Channel: In good condition, no debris collected. Minor shrinkage cracks observed on the abutment and counterfort wall.

3.3 Discharge Channel: In good condition. Ogee spillway showing some minor shrinkage cracks and bleeding.

3.4 Bridge and Piers: In excellent condition.

3.5 Gates and Operation Equipment: Gates are operational and were repainted 2 years ago. All equipment is maintained in good operational condition.

4. Reservoir:

4.1 Slopes: Gentle to moderate slopes bound the reservoir. All slopes appeared stable at the time of inspection.

4.2 Sedimentation: None observed.

5. Downstream Channel:

5.1 Condition: The stream channel is essentially clear except for the accumulation of sticks and minor debris where a fence crosses the stream - nothing serious.

5.2 Slopes: The west abutment is the only steep slope in the immediate area. This area consists of a discontinuous hill or ridge containing numerous shale outcrops. Shallow, gentle slopes were observed elsewhere.

5.3 Population and Facilities: No homes are within the floodplain for a distance of 4 miles downstream. At this point, the Maury River passes through the community of Wilson Springs, where there are two inhabited dwellings and numerous Park Department cabins that are used by campers on an intermittent basis. State Route 39 parallels the river for many more miles downstream.

This road is a primary highway that could be inundated during a dam break or during the PMF.

6 Instrumentation:

6.1 Monumentation: None

6.2 Observation Wells and Piezometers: No observation wells or piezometers were noted on the "as built" drawings and none were observed in the field.

APPENDIX IV

STABILITY ANALYSIS

Boy Scouts of America

1742 CONNECTICUT AVENUE NW • WASHINGTON, D C 20009 • DE 2-1962

Technical Review and Advisory Committee

February 11, 1965

National Capital Area Council
Boy Scouts of America
1742 Connecticut Avenue, N.W.
Washington, D.C. 20009

Attention: Mr. Elgin L. Deering, Director of Camp Development

Gentlemen:

Your Technical Review and Advisory Committee has reviewed the feasibility Report for the proposed dam and lake on the Council's Little Calfpasture River property.

The members of the Technical Review and Advisory Committee participating in review of this Report were:

D. Earl Jones, Jr., Chairman, Chief of Civil Engineering, Federal Housing Administration
Franklin F. Snyder, Chief of Hydraulics and Hydrology Division, Civil Works, United States Army Corps of Engineers
Harold O. Ogrosky, Chief of Hydraulics and Hydrology Division, United States Department of Agriculture, Soil Conservation Service
Jack Phelan, Assistant Chief Engineer, United States Department of Agriculture, Soil Conservation Service
Herbert Bossy, Chief of Hydraulics Research Division, United States Bureau of Public Roads
E. L. Hendricks, Associate Chief, Water Resources Division, United States Department of Commerce, Geological Survey
Joseph L. H. Paulhus, Manager, Water Management Information Division, United States Department of Commerce, Weather Bureau

This Committee, with the additional assistance of members of their individual staffs, to whom we are indebted, reviewed the proposal prepared by Herbert Associates, Inc., in depth. It is the consensus of the Committee that Herbert Associates has done an unusually fine and thorough job in drafting the Feasibility Report, with which the Committee is eminently well pleased.

The above-named Committee recommends acceptance of the Feasibility Report as drafted and further recommends that the Boy Scouts proceed from the Feasibility Stage to the Design Stage, using the firm of Herbert Associates, Inc.

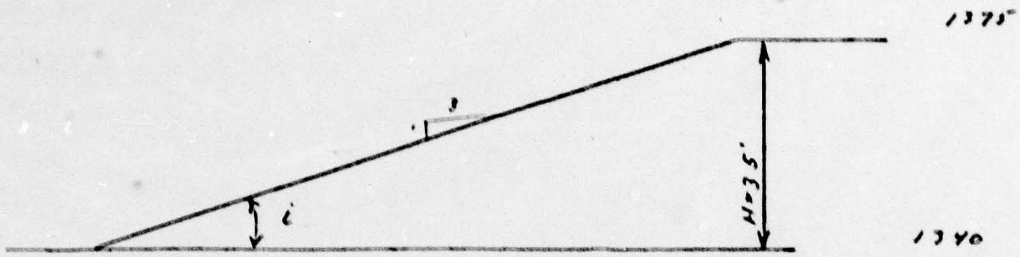
Sincerely,

NATIONAL CAPITAL AREA COUNCIL
BOY SCOUTS OF AMERICA

D. Earl Jones, Jr.

D. Earl Jones, Jr., Chairman
Technical Review and Advisory Committee

DEJ:ltn



KNOWN

$c = 900$ PSF $H = 35'$
$\phi = 23°$ $\gamma = 103.8$ (110.2 - CL - 97.7 - ML)
$i = 18°$
$\tan i = \frac{35}{105} = .333 = 18°24'$

ASSUMPTIONS

1. THRU THE TOE FAILURE
2. UNIFORM DENSITY ($\gamma = 103.8$)
3. LEVEL TOP
4. CONTINIOUS SLOPE

$$FS = \cot\alpha \cdot \tan\phi + \frac{c \cdot K_c}{\gamma H}$$

USED TABLES BY BERGER

Y COND

θ	β	$\cot\alpha$	K_c	$\tan\phi$	$\frac{c}{\gamma H}$	$\cot\alpha \tan\phi$	$\frac{c}{\gamma H} K_c$	FS
90°	14	4.51	7.34	.424	.248	1.93	1.82	3.75
	12	5.42	7.16	↓	↓	2.30	1.77	4.07
	16	4.01	7.99			1.70	1.98	3.68 ←
	18	3.78	9.41			1.60	2.33	3.93
80°	16°	3.85	8.51			1.59	2.11	3.70
	18	3.62	10.13			1.53	2.52	4.05
	14	4.31	7.72			1.84	1.91	3.75
100	14	4.79	7.12			2.03	1.77	3.80
	16	4.23	7.68			1.79	1.90	3.69
	18	3.98	8.92			1.69	2.21	3.90

MIN FS 3.68

HERBERT ASSOCIATES, Inc.
CONSULTING ENGINEERS
HARRISBURG, PENNSYLVANIA

Subject: SLOPE STABILITY
Job No.
Sheet No. 2 of 2
Drawing No.
Computed by ______ Checked by ______
Date: Nov 3, 64

COMPLETLY SATURATED

c = 900 PSF
ϕ = 23°
i = 18°

H = 35'
γ' = 103.8 − 62.4 = 41.4

θ	B	Cot a	K_c	tan ϕ	$\frac{c}{\gamma H}$	Cot a tan ϕ	$\frac{c}{\gamma H} K_c$	FS
90°	18	3.78	9.41	.424	.623	1.59	5.87	7.46
	16	4.01	7.99	↓	↓	1.70	4.98	6.68
	14	4.54	7.34			1.92	4.57	6.49 ←
	12	5.42	7.16			2.30	4.46	6.76
100°	14	4.79	7.12			2.03	4.43	6.46
	16	4.23	7.68			1.79	4.78	6.57
	18	3.78	8.92			1.69	5.55	7.24
80°	14	4.34	7.72			1.83	4.81	6.64
	16	3.85	8.51			1.63	5.30	6.93
	18	3.62	10.13			1.53	6.30	7.80
	12	5.20	7.49			2.20	4.66	6.86
100	10	7.28	7.23			3.09	4.50	7.59
	12	5.72	6.98			2.43	4.35	6.78

F.S. 6.49

SUDDEN DRAW DOWN

C = 900 PSF
ϕ = 23°
i = 18

H = 35'
γ' = 41.4

$\frac{\gamma'}{\gamma} = \frac{44.1}{103.8} = = .426 = w$

ϕw = 23° × .426 = 9.8°
= 9°48'

USE MIN

θ	B	Cot a	K_c	tan$_w\phi$	$\frac{c}{\gamma H}$	Cot a tan ϕ	$\frac{c}{\gamma H} K_c$	FS
90°	16	4.01	7.99	.173	.248	.69	1.98	2.67
	14	4.54	7.34			.79	1.82	2.61
	12	5.42	7.16			.94	1.78	2.72
100°	12	5.72	6.98			.99	1.73	2.72
	14	4.79	7.12			.83	1.76	2.59 ←
	16	4.23	7.68			.73	1.90	2.63
110°	12°	6.05	6.86			1.05	1.70	2.75
	14	5.07	6.97			.88	1.73	2.61
	16	4.47	7.47			.77	1.85	2.62

F.S. 2.59

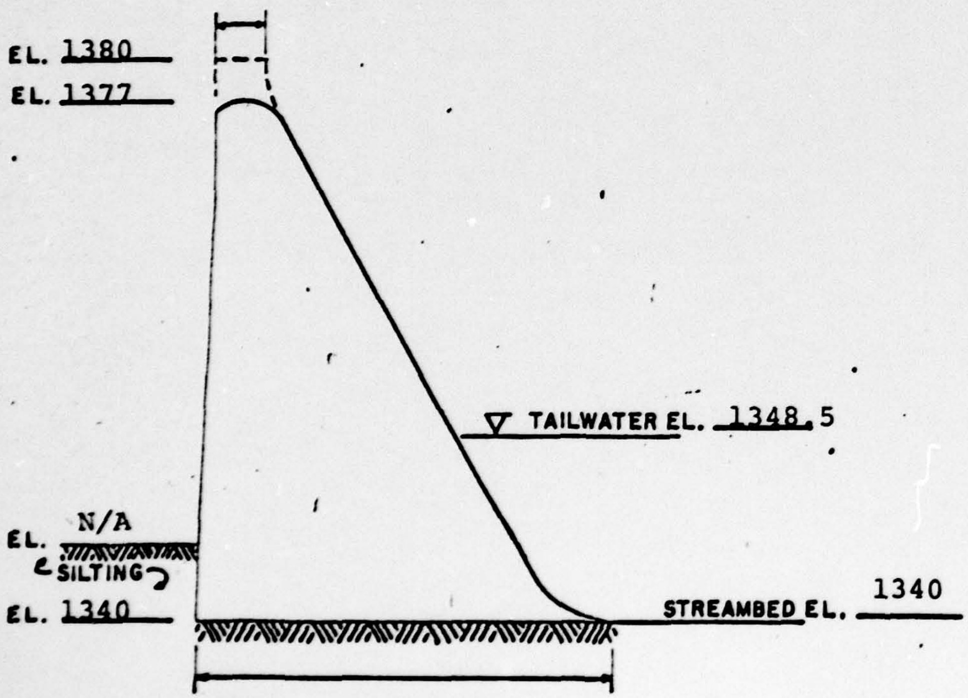
GRAVITY DAM DESIGN
STABILITY ANALYSIS

ANALYSIS DONE ON X FULL SECTION ___ PARTIAL SECTION

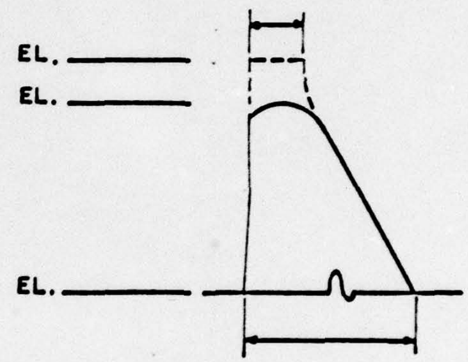
LOCATION OF SECTION ______________________

ANALYSIS PREPARED BY ______________________

LOADING CASE	ELEV. HEAD WATER	ELEV. TAIL WATER	ΣV.	ΣH	$\frac{\Sigma H}{\Sigma V}$	LOCATION RESULTANT FROM TOE	% BASE IN COMPRESSION	FACTOR SAFETY SLIDING	FOUNDATION PRESSURE	
									TOE ksf	HEEL ksf
EMPTY	-	-	75,306	0	0	25.7'	100	-	0.59	2.99
NORMAL POOL	1369	1340	44,956	26,290	0.58	23.5'	100	12.6	1.23	2.61
MAXIMUM FLOOD	1378.2	1348.5	55,276	22,716	0.41	20.0'	100	9.75	2.10	2.74



FULL SECTION



PARTIAL SECTION

NOTE: ANALYSIS DOES NOT CONSIDER ABUTMENT END RESTRAINT

REF: STABILITY CALCULATIONS BY HERBERT ASSOC., INC. HARRISONBURG, PA.

APPENDIX V

DESIGN REPORT AS PREPARED BY ORIGINAL DAM DESIGNERS PRIOR TO CONSTRUCTION OF DAM. (VERBATIM FROM ORIGINAL AS PRESENTED BY DAM DESIGN CONSULTANT)

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DESIGN REPORT AS PREPARED BY ORIGINAL DAM DESIGNERS PRIOR TO CONSTRUCTION OF DAM. (VERBATIM FROM ORIGINAL AS PRESENTED BY DAM DESIGN CONSULTANT)

CONCLUSIONS AND RECOMMENDATIONS

On the basis of the results of the investigations that have been conducted for this project, it is concluded that the site is suitable for the construction of an earth fill dam or a concrete dam. However, it is concluded that the site is best suited for the construction of an earthen dam.

The explorations along the dam axis and spillway show two stratum of soil overlying the black shale bedrock. The top five to six feet of these soils are silty clays and clayey silts with a low rate of natural permeability. Beneath these plastic soils is a stratum of silty sands with a high rate of permeability. This lower stratum with a high rate of permeability is undesirable as a part of a dam foundation because of its vulnerability to underseepage, excessive water loss and piping under the main portion of the dam. In view of this condition, it is recommended that the existing overburden soils be excavated to sound bedrock along the dam axis, and that a cut-off trench (using a base of about 14 feet and side slopes of 1:1) of impermeable soils be constructed to restrict seepage. It is very likely that the upper five to six feet of this excavated material (ML and CL) can be reused in the construction of the impervious section of the dam embankment. All topsoil must be stripped before storing this material for reuse. The

sandy soil should not be considered for the core or impermeable portion of the structure. It can, however, be used for other portions of the embankment, as subsequently discussed.

The shale bedrock was found to be weathered for the first two or three feet of depth from its surface. This material should also be excavated to such depths that the sound rock is exposed. The total excavation to sound rock appears to be to elevation 1340.

The abutments on both the east side and the west side of the dam appear to have good shale to tie into. On the west side, the topography of the existing ground surface rises sharply from elevation 1350$\pm$ to 1416$\pm$ in 100 feet. Here the abutment may have to be carried nearly 50 feet into the hillside before hitting sound rock. On the east side of the dam, where the greatest depth of overburden exists, the topography of the area rises more gently and it appears that that the abutment will have to be carried to Station 19+00, or perhaps to Station 21+00, in order to tie into sound rock.

The cut-off trench excavation on the east side will be deeper than that required along the main portion of the dam due to the deep deposits of cobbles and boulders. In this area, the excavation may go as deep as 25 feet before reaching sound rock. The preliminary considerations for the placement of the spillway indicate that its most suitable position will be on the east side of the dam. In this case it is imperative that the excavation be carried to sound rock to provide a suitable foundation for the spillway section.

With regard to the load carrying capacity of the bedrock at this site, there is no problem. This formation is more than capable of carrying reasonably heavy loads that might be developed by the dam embankment or the spillway. However, the allowable bearing capacity used for design should not exceed 20 TSF. Furthermore, the rock should provide a stable foundation for the entire structure under all conditions of loading and saturation.

The principal problem for this project is the apparent inability of the rock structure to resist the flow of water under pressure to the degree that excessive leakage or loss of water will be prevented. These conclusions are based on the results of the field pressure tests which were conducted on the in-place rock strata. The average water loss for all tests indicate a permeability value of 1000 feet per year or 10^{-3} cm. per second. In accordance with the design practices of the Bureau of Reclamation, this rate of permeability is classified as pervious (10^{-6} cm/sec. - impervious; 10^{-4} cm/sec. - semi-pervious; over 10^{-4} cm/sec. - pervious). This rate of water loss is due primarily to the type of rock (sedimentary shale), its stratification, and the presence of calcite lenses between the sedimentary layers. Dissolution of the calcite layers will foster leakage paths, as will the shifting and slipping of the bedding planes during the folding of this area in the geologic past. Joints, fissures and crevices are all possible contributors to the pervious condition of the formation. In view of these facts, it is recommended that a systematic program of grouting the rock foundation be developed and carried out at this site to improve the water holding capacity of the foundation. Grouting under pressure to seal seams,

joints and other openings is usually carried to a depth below the rock surface equal to the reservoir heads above the surface of the bedrock. Indications are that a maximum elevation for the lake level would be near elevation 1370. On this basis, with the average rock surface at elevation 1340±, the grouting operations should be carried to a depth of about 30 feet below the rock surface or to elevation 1310±, following a split spacing, stage grouting zone method (15 feet per zone). Depending on the nature of the rock surface after excavation to the foundation base is completed, a grouting cap approximately 3 feet x 3 feet in dimension may be required to facilitate the subsurface grouting.

Careful consideration must be given to the proper design of the abutment contacts, particularly at the east end where the tie-in points may have to be extended several hundred feet beyond the water level contact with the ground surface because of the rather thick deposit of overburden in this area and relatively gentle slope of the bedrock surface. All loose or overhanging rocks or unsuitable materials must be removed and the tie-in slopes reduced so as to have proper bond of the embankment materials to the abutment materials.

All indications are that the entire dam embankment, with the exception of the rip-rap on the upstream face, can be almost constructed of one basic material inasmuch as a sufficient quantity of borrow material is available from the proposed borrow area and since all of the soils from this area are suitable for the construction of the dam. The average coefficient of

permeability of the borrow soils when compacted to at least 95% of the maximum dry density at optimum moisture content is 3.4×10^{-7} cm/sec. This value is considered impervious and is suitable for the design of the core of the dam.

Even though there is a sufficient quantity of borrow material, the earthen embankment should be of the zoned type to take advantage of, and make use of, the virgin materials recommended to be excavated from the dam area. The impervious inner section should be constructed of soil obtained from the borrow area, while the ML and CL soils excavated from the dam section should be used in the formation of the upstream section. For the downstream portion of the embankment, the SM, SC, and SP soil types, the boulders, cobbles and weathered rock, all as shown on Plate No. 4 in (Appendix E), should be used, supplemented by materials from the borrow area as may be required. Special features of the dam, such as drains and filters, will necessarily require other materials.

Because it is impracticable, or sometimes impossible, to stop all seepage under a dam, provision must be made to contain the possible displacement of silts or clays due to the escape of water under pressure at the downstream toe by providing downstream inverted filters.

Relative to the slope design of the embankment, a cursory study of the stability of the slopes indicates that slope ratios of 3:1 will be suitable for this project. This analysis has used the values of cohesion and friction

as determined by the laboratory direct shear tests: cohesion 900 psf and a friction angle of 23°. Draw-down conditions were not considered during this study.

The slopes of the impervious core section should be 1/2:1, while the abutment contact slopes should be no steeper than 1:1.

Since the excavation to reach the foundation level will be below the ground water level, as recorded during September 1964, provisions should be made to control and remove all free water from the area during construction. The ground water level is near elevation 1343, and the foundation level is near elevation 1340±.

In summary, the following recommendations are made for the design and construction of the proposed dam:

1. That all of the overburden soils, together with the weathered portion of the shale bedrock, be excavated to firm bedrock along the dam axis, including the abutments, and a cut-off trench of impermeable soils be constructed.

2. That the spillway section be located on sound bedrock.

3. That the existing virgin soils of the entire dam area, outside of the cut-off trench sector, upon which the embankment will rest, be proof rolled with a 50-ton

pneumatic tired roller before receiving any fill materials, so as to increase the soil-supporting and impermeability characteristics of the soil strata. Any soft areas disclosed by this rolling should be removed and replaced with suitable material.

4. That all the materials excavated from the dam and spillway area, with the exception of topsoil and organic materials, be reused in the construction of the embankment as previously discussed.

5. That the rock stratum forming the dam foundation and abutments be pressure grouted to a depth of 30 feet below the rock surface to seal all joints, cracks, etc.

6. That the dam embankment be constructed using the basic on-site materials available from the proposed borrow area, and that these soils be placed under controlled supervision to a minimum of 95% of the maximum dry density, as determined by the standard compaction tests, ASTM Designation D-698-58T.

7. That the following shear strength values be used for the compacted embankment in the design of stable slopes:

Cohesion: 900 PSF

Friction: 23°

8. That the coefficient of permeability for design purposes be taken as 3×10^{-7} cm/sec. at 20°C.

9. That the upstream slope of the dam be faced with rip-rap made up of locally available fine grained sandstone, and that the downstream slope be protected against erosion by topsoiling and seeding.

APPENDIX VI- REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, Department of Army, Office of the Chief of Engineers, 46 pp.
2. Design of Small Dams, U. S. Department of Interior, Bureau of Reclamation, 1974, 816 pp.
3. Geology of the Lexington Quadrangle, Virginia, Reports of Investigations No. 1 , K. F. Bick, Virginia Division of Mineral Resources, 1960, 40 pp.
4. Section 4, Hydrology, Part 1 Watershed Planning, SCS National Engineering Handbook, Soil Conservation Service, U. S. Department of Agriculture, 1964.
5. Hydrometerological Report No. 33, U. S. Department of commerce, Weather Bureau, U. S. Department of Army, Corps of Engineers, Washington, D. C., April 1956.
6. Technical Paper No. 40, U. S. Department of Commerce, Weather Bureau, Washington, D. C., May 1961.
7. Siltation and Eutrophication in Lake Merriweather, Independent Study, GY401, Mathew W. Delfeit. Undated, approximately 1966.